THE EFFECTIVENESS OF APPLYING DYNAMIC LANE ASSIGNMENT AT ALL APPROACHES OF SIGNALIZED INTERSECTION

BY

Muath Marwan Najjar

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This thesis, written by Muath M. Najjar under the direction his thesis advisor and approved by his thesis committee, has been presented and accepted by the Dean of Graduate Studies, in partial fulfillment of the requirements for the degree of **MASTER OF SCIENCE IN CIVIL ENGINEERING.**

Dr. Wael Alhajyaseen
20/5/2015

Dr. Wael Al-Hajyaseen
(Advisor)

Dr. Omar Abdullah Al-Swailem

Department Chairman (A)

Prof. Nedal T. Ratrout
(Member)

Prof. Salam A. Zummo
Dean of Graduate Studies

عمادة الدراسات العليا
DEANSHIP OF GRADUATE STUDIES

Dr. Khalaf A. Al-Ofi
(Member)

26/5/15
Date

بسم الله الرحمن الرحيم

((قُلْ إِنَّ صَلاتِي وَنُسُكِي وَمَحْيَايَ وَمَمَاتِي لِلَّهِ رَبِّ الْعَالَمِينَ))

Dedication to

My beloved parents, my brothers, my sister

My Holy Homeland Palestine

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All praise and glory be to Allah for his limitless help and guidance. Peace pleasing of Allah be upon His prophet Mohammed.

I would like to express my profound gratitude and appreciation to my advisor Dr. Wael Al-Hajyaseen for his guidance and critical help from beginning of this study till the end. His encouragement, guidance and invaluable suggestions enabled me to develop an understanding of the subject.

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LIST OF ABBREVIATIONS

DLG	:	Dynamic Lane Group
MOE	:	Measures of Effectiveness
ATDM	:	Active Traffic and Demand Management
ATM	:	Active Traffic Management
PHF	:	Peak Hour Factor
DLM	:	Dynamic Lane Management
VPH	:	Vehicle per Hour
HCM	:	Highway Capacity Manual
FLG	:	Fixed Lane Group
LGC	:	Lane Group Combination
D	:	Intersection Delay
MAPE	:	Mean Absolute Percentage Error

ABSTRACT

Full Name : [Muath Marwan Mahmoud Najjar]

Thesis Title : **[The Effectiveness of Applying Dynamic Lane Assignment at All Approaches of Signalized Intersection]**

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Intersection performance is affected by the spatial variations in the traffic demand. Existing traffic control strategies at signalized intersection assume a fixed movement per lane (lane group) for each approach. However, it is observed that some intersection, especially in urban areas, have a significant variations in traffic demand among different movements of the same approach (LT, TH, RT). Under this condition, adopting a fixed lane group strategy will result in the waste of time-space resources. One of the applications of Intelligent Transportation Systems (ITS) solution at isolated intersections is to apply Dynamic Lane Grouping (DLG) in order to improve their mobility performance. The main objective of this research is to evaluate the effectiveness of applying DLG in identifying the optimal Lane Group Combination (LGC) and optimized signal timing at isolated signalized intersection based on minimum intersection delay criterion. It was concluded that applying DLG along with signal optimizing can enhance the performance of the signalized intersection by reducing the intersection delay which leads to reduce the queue length and the amount of fuel consumption and emissions. Furthermore, applying DLG for all approaches resulted in a significant reduction in cycle length and the associated intersection delay.

ملخص الرسالة

الاسم الكامل: معاذ مروان محمود نجار

عنوان الرسالة: دراسة فعالية تطبيق تقنية المسار المتغير بشكل ديناميكي على كافة اتجاهات المفترق الخاضع لتحكم الاشارة الضوئية

التخصص: هندسه مدنية

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يتأثر اداء الاشارات المرورية على التقاطعات بشكل ملحوظ باختلاف اتجاه الحركات خلال اليوم وزيادة هذا الاختلاف يؤدي الى تأخير السيارات مما يتسبب بحدوث ازدحامات مرورية خانقة وبالتالي يؤدي الى زيادة التلوث الناجم من انبعاثات عوادم السيارات. تتلخص هذه الدراسة في تطبيق الية جديدة بحيث تجعل استخدام امثل للاشارة الضوئية وذلك من خلال تغيرها ديناميكيا لتوفر السعة المطلوبة لكل حركة من الحركات (الاتجاه نحو اليسار, اماما , نحو اليمين). اظهرت نتائج الدراسة التأثير الايجابي الملحوظ في تطبيق الالية الجديدة عوضا عن استخدام الالية القديمة (التحديد المسبق لاتجاهات المسارب خلال التقاطع). إن تطبيق هذه الالية يؤدي الى تقليل التأخير وبالتالي تقليل التلوث والحصول على افضل طول اشارة ضوئية لذلك التقاطع بحيث يخدم اي اختلاف في اتجاه الحركات المرورية خلال التقاطع مما يزيد كفاءة الاشارة الضوئية.

CHAPTER ONE

INTRODUCTION

1.1 Background

Severe traffic safety problems and increasing congestion along with higher emission and energy consumption are the main challenges which are being faced by our society. One of the main reasons is the continuous growth in traffic demand which cannot be met by the limited capacities of existing roadway facilitates. Intelligent Transportation Systems (ITSs) have been receiving increased interest since they can enhance traffic operations and lead to a significant gain in mobility and sustainability as well.

One of the main ITS successful applications is Active Traffic and Demand Management (ATDM) strategy which has become a powerful tool as a possible solution to the growing congestion problem [1]. Signalized intersections are the main facilities that ATDM has been focusing along arterials [2]. Conventional signal control strategies assume fixed lane utilization on intersection approaches. Spatial variations in traffic demand degrade intersection performance. It can result in the waste of time-space resources caused by the increasing time of the green light for cars using the same phase and going to the opposite direction. A more rational and reasonable strategy can provide a better time-space allocation at all roads approaching the intersections by dynamically assigning more lanes to high demand movements; this is called dynamic lane assignment strategy or Dynamic Lane Grouping (DLG). Such strategy will result in significant improvement in the performance of signal control (e.g. significantly lower delays) since lane allocation will be dynamically performed in response to real-time movement demands.

It is important to mention that this work (DLG) has been performed for one approach only as a part of a project (IN 131009) of Deanship of Scientific Research (DSR) at KFUPM. This study aims to investigate the effectiveness of applying dynamic lane grouping at four-leg signalized intersections. The following sections will identify the significance of the study, problem statement, the objectives of this research and thesis organizations.

1.2 Significance of the Study

Signalized intersection is key elements along transportation network. These points impose large delays on vehicles when significant fluctuation in traffic demand per movement in each approach exists since the allocated space per movement is fixed. This increases the aim to improve the operation at signalized intersections by using a new technique, DLG, for all approaches of the signalized intersection. This research will discuss the new technique which has not been discussed for the whole intersection before. Thus, the application of new strategy (DLG) that allows a flexible space allocation per movement based on traffic demand is very advantageous. It is expected that DLG will lead to a significant reduction in intersection delay and therefore a drastic improvement in intersection performance.

1.3 Problem Statement

Operations of signalized intersections considerably affect the performance of the whole road system and further leave impacts on environment and safety. The main conventional signal control methods include stage-based and group-based approaches. In the stage-based approach, compatible traffic movements are grouped to move together in a specific time span within a signal cycle, which are referred to as stages, and green times are then given to each stage. In contrast, the group-based approach directly allocates green times to traffic movements without the necessity to maintain a specific stage structure. Thereby, the group-based control strategy is more flexible to generate complicated and unexpected phasing plans for intersection users [3]. Both of the control approaches optimize the cycle length and phasing assuming a fixed lane configuration regardless of the traffic demand. However, due to the limited lane capacity and the significant fluctuation in the relative traffic demand between different movements as shown in Figure 1.1, which is widely observed at most signalized intersection in Al-Khobar and Dammam metropolitan areas, it is more likely that intersections become oversaturated and thus signals fail to serve vehicles without suffering substantial congestion. Dynamic lane assignment is proposed to have significant positive effect on the performance of the intersection. For example, long queue will occur especially for left-turn vehicles as shown in Figure 1.2a when there is an exclusive left turn lane associated with high volume of left turning vehicles due to the fluctuation in traffic demand within the day. However, in reality drivers illegally tend to use the through lane to make their left turning movement. It is unsafe to perform this kind of movement since it leads to side swipe crash Figure 1.3 and confusion between drivers. The confusion is due to the sudden change in movement from through to left.

Also, this kind of behavior will have a negative effect. The negative effect, like reducing the capacity of the intersection area and blockage for the through traffic, is due to the number of receiving lanes for the LT movement.

Dynamic lane assignment is proposed to solve these kinds of problems since it will provide improved lane utilization by dynamically making the second lane shared with left instead of exclusive through as shown in Figure 1.2b which will provide better time-space allocation and increasing the capacity for the left turn movement as well. Furthermore, DLG will also accommodate the drivers' illegal behavior.

Few existing studies addressed the application of DLG either at one lane of chosen approach or at one approach of signalized intersection assuming fixed cycle length. So, it is important to investigate the effectiveness of applying DLG for whole approaches of the signalized intersection. Also, signal timing optimization will be performed as combined with the DLG technique.

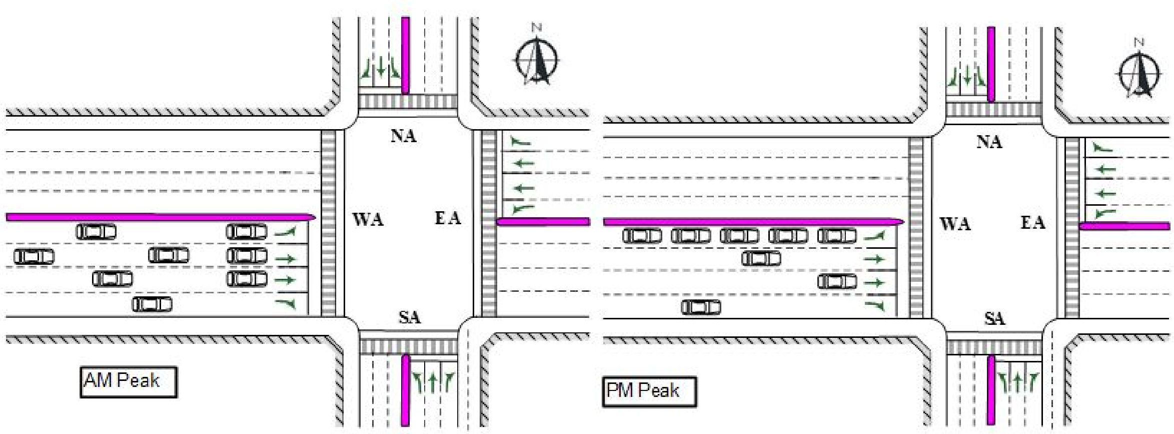
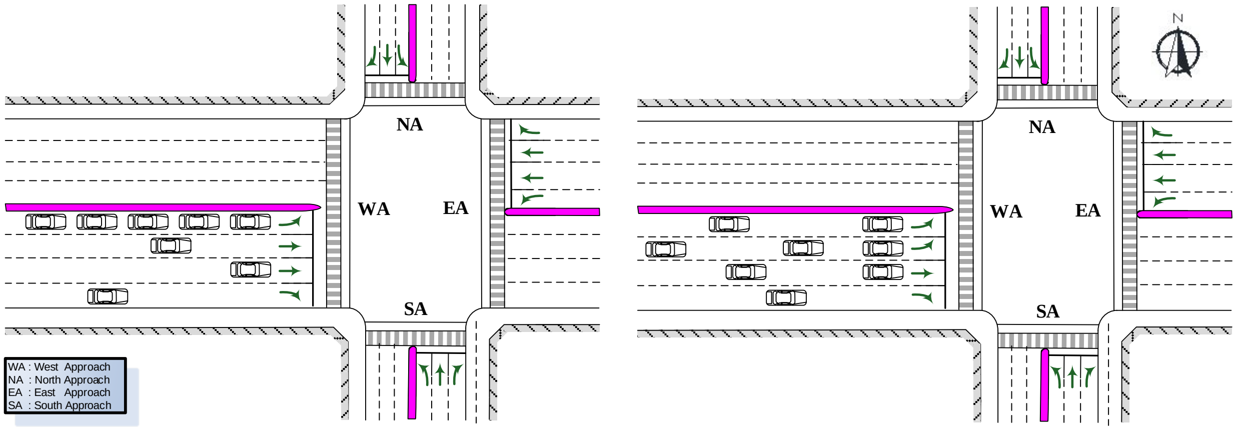


Figure 1.1: Spatial demand variation



Poor lane utilization
a

Improved lane utilization
b

Figure 1.2: Lane utilization

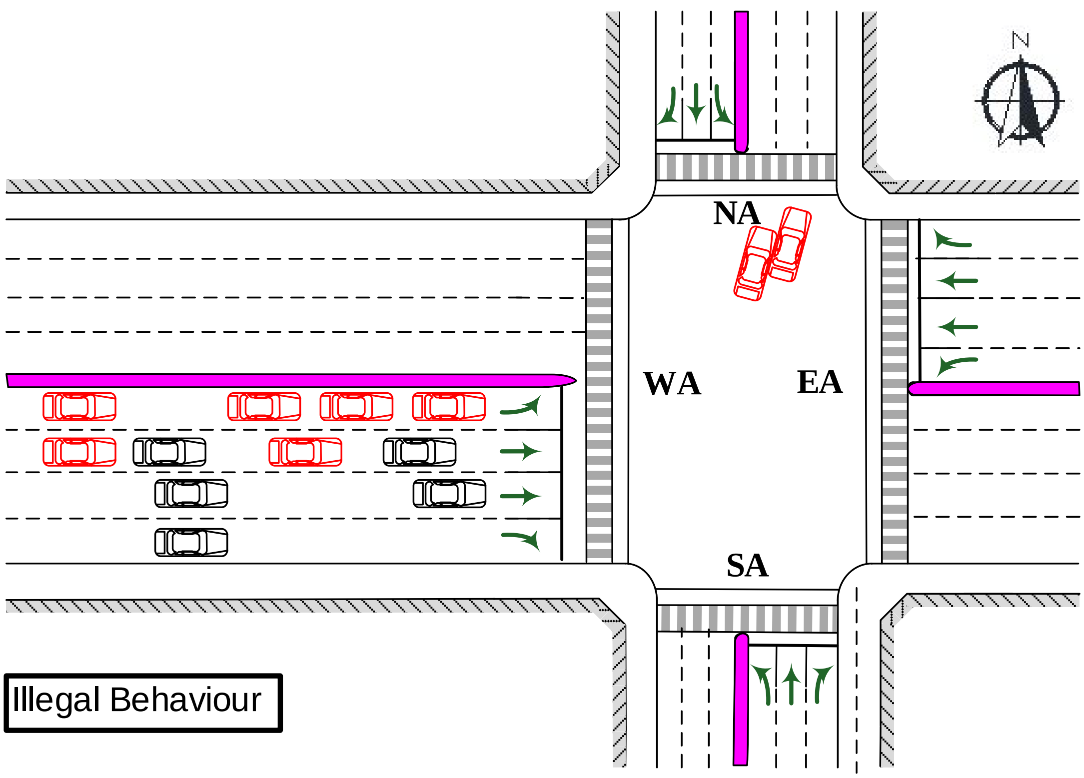


Figure 1.3: Illegal behaviour of the LT drivers

1.4 Research Objectives

The main goal of this study is to investigate the effectiveness of applying Dynamic Lane Grouping (DLG) at all approaches of a signalized intersection combined with the optimization of signal timing parameter (Cycle length, green splits, etc.)

To reach the main goal, there are several objectives to achieve:

1. Development of the analytical model structure to identify the optimal lane group and optimum cycle length based on minimum intersection delay. In this part, the estimation method of the optimum cycle length, volume to capacity ratio and intersection delay will be determined. The output of the model, optimized cycle lengths and calculated delays will be compared with those estimated by traffic simulation programs such as Vistro/Vissim, Synchro/Simtraffic.
2. Comparison analysis. To verify the effectiveness of the proposed dynamic lane grouping strategy. Estimated delays after applying the proposed strategy will be compared with the delays estimated assuming Fixed Lane Grouping (FLG).
3. Sensitivity analysis and model verification. The sensitivity of the proposed strategy to the variation in the relative demand between different movements in the same approach and to the variation in the relative demand between different approaches will be analyzed. To verify the effectiveness of the model in real world, field data of traffic at a signalized intersection will be collected to assess the performance of DLG over fixed lane strategy.

1.5 Thesis Organizations

This thesis is organized as follows: Chapter 2 presents an intensive literature review on the optimization of signal timing and the application of DLG. It explains the positioning of this study between existing works. Chapter 3 includes the development of the model structure with detailed explanation about each sub-model. Chapter 4 presents the sensitivity analysis and model verification for the enhancement of DLG over FLG. Chapter 5 shows a comparison between the results of the developed model (DLG) with well-known traffic software such as Synchro7 and Highway Capacity Software (HCS2000). At the end, chapter 6 draws the conclusion and recommendations for the future works. This organization is shown in the following flow chart.

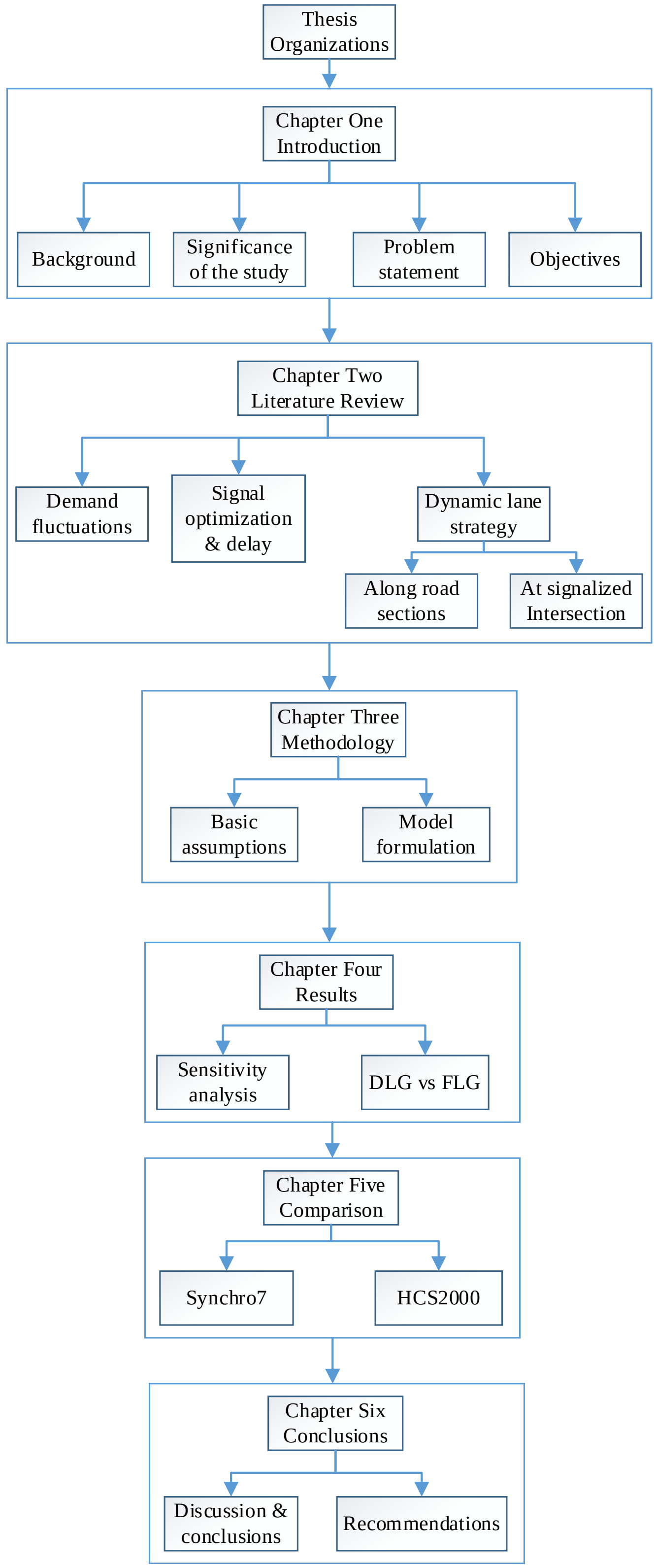


Figure 1.4: Thesis flow chart

CHAPTER TWO

2 LITERATURE REVIEW

In this chapter an intensive review of existing studies addressing demand fluctuations, signal optimizing, and DLG along road sections and at signalized intersection is presented to understand the state of the art and the positioning of this study among previous ones.

2.1 Demand Fluctuations

Traffic planning, operation, and control have always critical concern in the variation in traffic origin–destination demand. A considerable variability in day-to day or within-the-day traffic demand may be experienced at signalized intersection [4]. From 11 intersections in Milwaukee, Wisconsin using weekday data, the Coefficient of Variation (CV) in peak hour traffic volume was examined and found that it ranged from .048 to .155. These values were applied to volumes at a simulated intersection to quantify their impacts on service levels. it is concluded that traffic volume variations deteriorate service levels at intersections [5].

Tarko and Perez-Cartagena studied the variation in traffic demand using 45 intersections in the Indiana state, USA. They investigated both the temporal and spatial variations in Peak Hour Factor (PHF) and concluded that the day-to-day variability of PHF was found to be as strong as the site-to-site variability [6]. Zhou et al. by using an empirical study revealed that the patterns of traffic demand varied considerably in both temporal and spatial dimensions [7].

Previous studies on the fluctuation in traffic demand lay a basis for the concept of DLG, which aims at adjusting lane allocation online to respond to the spatial variation in traffic demand.

2.2 Signal Optimization and Delays

One of the first work in signal control is referred to Webster [8] who establish a formula that determine signal control setting for an isolated intersection. Based on the assumption that the traffic volume did not exceed the intersection capacity, a predetermined desired degree of saturation was given to each approach to limit the level of saturation. The goal for was to minimize the total intersection delay which is subjected to some constraints during the entire period of operation like cycle length and splits. Conversely, the model did not respond to the flow variation. So, the signal control may be far from optimal control [9]. This problem can be partially solved using traffic actuated control by extending green phases in response to real time arrivals of the movement being served. However, the extension of the current green phase may not stop if there are long queues on other movements [10, 11]. In this research, the model will consider the fluctuation in the traffic demand per movement of each approach which can be used later as a part to make a special adaptive signal control that adjusts the signal timing parameters in response to real time traffic variations.

One of the main Measures of Effectiveness (MOE) is the delay which is used to evaluate the effect of traffic management, reflects the travelling quality of traffic participation fundamentally. Even though certain research efforts have been achieved on delay at timing signal control intersection [12-14], researchers and practitioners define delay in many ways: stopped delay, control (signal) delay, travel-time delay, queue delay, and others. Stopped delay is the time an individual vehicle spends stopped in a queue while waiting to enter an intersection [15]. Average stopped delay is the total stopped delay experienced by all vehicles arriving during a designated period divided by the total volume of vehicles arriving during the same period.

2.3 Dynamic Lane Strategy

In urban areas, congestion is the main consequence of the ever-increasing traffic demand. The variability in volumes of turning movements aggravates the enduring problem of congestion. Many studies have been done in order to solve this problem taken into consideration different measures of effectiveness for evaluation of dynamic lane assignment such as queue length, total delay and saturation flow rate at various transportation facilities such as freeways, arterials and isolated intersections [16-19].

The following subsections explain the background of using dynamic lane strategy in road sections and at signalized intersections.

2.3.1 Dynamic Lane Strategy along Road Section

To increase the capacity of road cross-section, the concept of Dynamic Lane Management (DLM), as a congestion relief scheme, has intensively been applied in freeway operation through the opening of hard shoulders to traffic when demand is high. This policy proved to have significant effects in reducing travel time and improving safety [19].

Empirical observations in Hessen, Germany, show that in addition to safety improvements after the application of a DLM strategy which is basically based on dynamic opening of the hard shoulder to traffic during peak periods, operating speed increases and travel time loss decreases [17]. Also, Cohen [20] shows that using shoulder as a lane, improves the capacity and reduces the emissions.

Dutch experience over 160 km motorway segments, suggests that dynamic management of the hard shoulder operation during peak periods is 2.5 times more cost effective than constructing new infrastructures. Consequently, traffic throughput was found to be increased by 7% to 22% after opening the hard shoulder to traffic on peak periods [16].

The UK Highways Agency implemented an Active Traffic Management (ATM) system as a pilot scheme over the 17 km stretch of the M42 highway (3 lanes + hard shoulder) that allows the operators to dynamically open the hard shoulder to traffic at busy hours of the day [18]. A before-after study pinpointed significant improvements in peak period travel conditions [18]. Moreover, travel times were reduced by an average of 24% (northbound) and 9% (southbound). These studies mainly concentrated on the road section and road network.

2.3.2 Dynamic Lane Strategy at Signalized Intersection

Many studies were performed to evaluate the effectiveness of some techniques rather than dynamic lane assignment in reducing the congestion and increasing the capacity at signalized intersection [21-25].

In a recent study, Zhang and Wu [26, 27] analyzed the effects of DLG using PARAMICS simulation software at a hypothetical isolated signalized intersection assuming predefined demand levels along with fixed cycle length. In the analyzed scenario, one approach only had variable traffic demand and dynamic traffic assignment as well. It was concluded that the DLG strategy improves the mobility performance in terms of reduction in average vehicle delay and number of stops. Furthermore, these benefits increase as the traffic volumes for the different turning movements deviate from the baseline demand pattern. The same benefits were achieved in fuel consumption and emissions. Presented analysis was performed on a hypothetical intersection assuming large fluctuation in traffic demand at one road approach.

Another model was developed by Ding et al. [28] to optimize lane use and signal timings for isolated signalized intersections with dynamic lanes which can be used for different movements. This model was evaluated based on its ability in minimizing the intersection delay. Only one variable lane can be provided on each approach was used as assumption for this model. The result of the model shows a reduction in the average delay and improvement in the efficiency of the intersection. However, this model was applied at one lane only.

Zhong et al. [29] analyzed the impacts of dynamic lane assignment upon the time allocation at an approach of signal control intersection. An optimization model based on time-space resource combination was proposed. Through numerical analysis, it is concluded that this method produces optimum benefit scheme based on dynamic lane functional partition within a given traffic demand range. This optimum scheme showed significant decrease on traffic delay. However, it is not certain that these results will be valid if the dynamic lane function optimization method is extended to a whole intersection, which was not investigated.

Here comes this study to complete previous works by applying dynamic lane group at all approaches of an intersection combined with signal phasing optimization as well.

In all previous dynamic lane management experiences, Intelligent Transportation Systems (ITS) such as VMS can be used as communication tools by road operators to inform road users about operational conditions such as speed limit, lane configuration and so on. Variable message signs are increasingly used in the transportation sector to give dynamic information in order to improve and make the journey more efficient and safe. One of the principal issues with information is what kind of information is provided. Several works showed that the effects of traffic information can be varied with information provision strategy [1].

Webster's [8] and other numerous methods for signal optimization at intersections focus on reducing vehicle delays by checking various phasing patterns assuming a fixed lane configuration. In most cases, based on peak hour demand, traffic lanes will be assigned to different movements at each approach of the intersection. However, due to the fluctuation in the relative traffic demand between different movements at the same approach, the signal optimization process may result in long signal cycle durations assuming fixed lane groups, which deteriorates the overall mobility levels of signalized intersections and might lead to risky vehicle and pedestrian behaviors [30].

Almost all of the previous studies are about applying dynamic lane grouping on one approach only. This research will concentrate on applying this technique on the whole approaches of signalized intersection.

The principle of dynamic utilization of lane resources has been proposed in various forms. In freeways management, dynamic lane allocation has been developed in the Netherlands for traffic segregation [31]. For signalized intersections, developed concepts include a system for dynamic lane assignment tested in Houston, Texas [32]; a dynamic left-lane concept in the Netherlands [33]; and dynamic lane-use management [34], among others. A similar concept was also studied within the framework of fully automated intersections with autonomous vehicles [35].

Since all previous studies either focus on applying DLG using fixed cycle length or optimizing signal timing using fixed lane group, this study will combine them together in one model to find the optimum lane group along with best signal timing.

2.4 Traffic Software's Comparison

Benekohal et al. conducted a study to compare between HCS, Synchro, PASSER and CORSIM at urban arterials. The comparison was mainly based on the amount of the delay reduction materialized when optimized signal settings is implemented. It was concluded that Synchro delays were significantly different than the delays before optimization [36].

Trueblood [37] conducted a study to compare between CORSIM and SimTraffic on an arterial with low to moderate traffic. The comparison was based on some important MOEs such as delay, number of stops, LOS and queue length. They found that both models resulted in very close values of MOEs. Another study which was conducted by Choa et al. [38] tried to compare between three microscopic simulation tools which are CORSIM, PARAMICS, and VISSIM. The comparison was based on different factors such as model development (i.e. input requirements and coding effort), calibration to field conditions (i.e. driver behavior, traffic flow characteristics and traffic control operations), validation requirements (i.e. travel times, queue lengths and level of service), animation (i.e. graphics, viewing options, and backgrounds), and model output and consistency with the HCM 2000. They concluded that CORSIM outperformed others due to the least difficulty in coding and its ability to compute control delay for individual approaches. The simulations of PARAMICS and VISSIM, along with their 3-D capabilities, were more closely reflected the actual conditions.

Tian et al. [39] made a study to compare between CORSIM, SimTraffic and VISSIM at a signalized intersection. The comparison was mainly based on the variation in capacity ratio and delay. Different scenarios of traffic were assumed at the intersection. It was found that CORSIM produces the lowest variation in both delay and throughput, whereas SimTraffic produced the highest variation especially when the volume approached capacity. Kosman et al. [40] compared between VISSIM and CORSIM in terms of project level emission modelling. They found that either model may perform adequately for estimating average speed as input for emission analysis provided that proper validation is adopted.

CHAPTER THREE

3 METHODOLOGY

This chapter discusses the principles, assumptions and methods used to build the DLG model using MATLAB environment. For a better understanding, an intersection with specific layout is selected to demonstrate the model development. Instead of having a hypothetical intersection an existing site that suffers from significant demand variations is selected. The site that used to perform the analysis is the intersection of Abu Obaida Street with Prince Faisal Street located near IEKA, in Dhahran, Saudi Arabia as shown in Figure 3.1. It consists of four lanes in the west-east (WA-EA) approaches and three lanes in the north-south (NA-SA) approaches. i represents approach number starting from one for the west approach and moving clockwise. k represents lane number at approach i staring by the far left lane (near the median) and ending with the outer shoulder lane. The study site is operated with four phases signal plan where each approach is assigned to a phase (Figure 3.2).

Figure 3.1: Geographic map for the intersection site

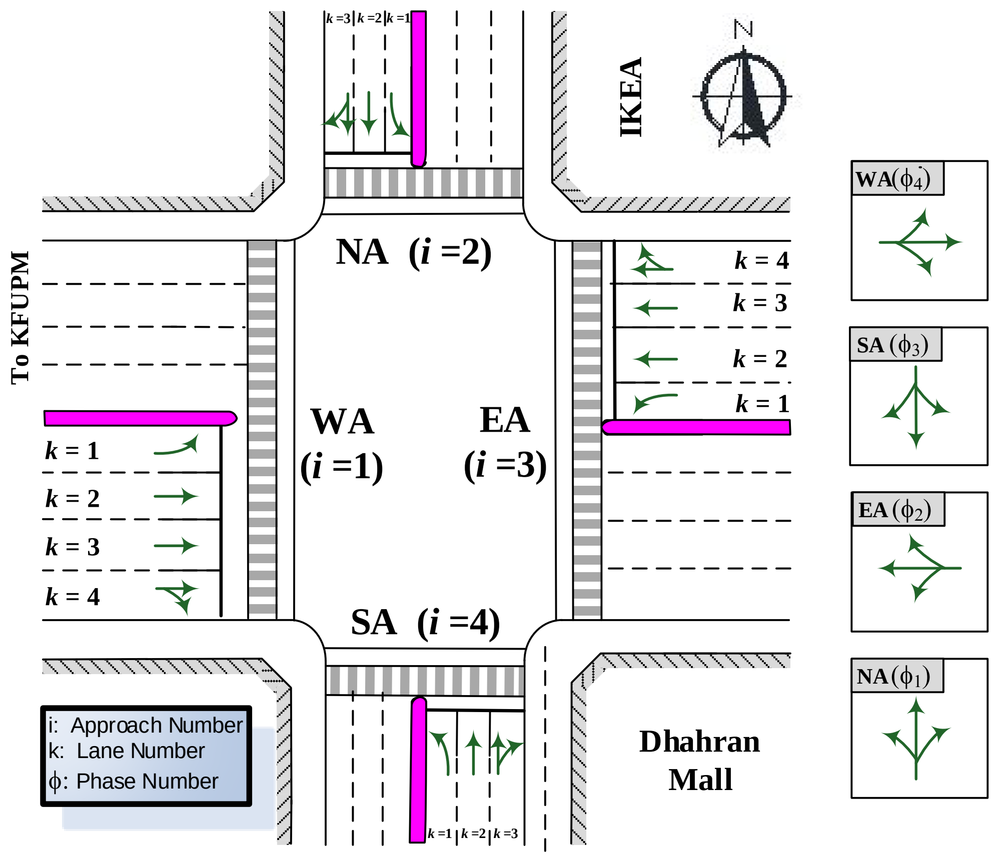


Figure 3.2: Intersection layout

3.1 Basic Assumptions

The following assumptions have been made in order to simplify the problem formulation of DLG strategy.

1. The study investigates the application of DLG at isolated intersection only. The applicability on a network or coordinated intersection level is not addressed.
2. The intersection is characterized with large variation in the directional traffic demand left-turning (LT), through (TH) and right-turning (RT) during the day (tide-type of traffic flow).
3. This study assumes the cycle length is not fixed. Optimum cycle length will be estimated for each traffic demand combination based on the minimum average intersection delay.
4. Four-phase signal plan is assumed for the signal operation in which each approach is assigned to separate phase. Based on this phasing, all LT are protected. No permissive LT movement exists.
5. Since U-turns are not common at all intersection, the model assumes no U-turn. However, the model can be easily modified to incorporate this movement.
6. The lane selection principle assumes that saturation flow ratios for the adjacent lanes which share turning movements are equal.
7. In the delay calculation, it is assumed that there is no initial queue delay from the previous analysis period.

It is important to note that the implementation of DLG needs specific advanced technologies like fiber optic lane indication sign [32], variable message sign [41], pre-signal [42] and connected vehicle technology.

The considered performance measures in this study to evaluate the benefits of DLG are the average delay per vehicle for the whole intersection (D) and maximum lane volume-to-capacity ratio for each approach. The main objective function of the developed optimization model is the minimization of the average intersection delay (D).

3.2 Model Flowchart

The model is built using MATLAB environment. Figure 3.3 shows the structure of this model which contains three dependent loops. The center loop is to identify the optimal cycle length for a specific lane group and specific demand combination. The second loop is to identify the optimum lane group for a specific demand combination. The last loop, which is the largest loop, is to change the input demand combination by which the first and second loop will run again. The main input parameters for the model are traffic volumes for all movements and number of lanes in each approach. The main output of the model for each demand combination is the optimal lane group combination LGCo based on the minimum average intersection delay D. Furthermore, the optimum cycle length of the optimal lane group will be determined. Using the developed model, the effectiveness of DLG will be assessed and compared with FLG strategy.

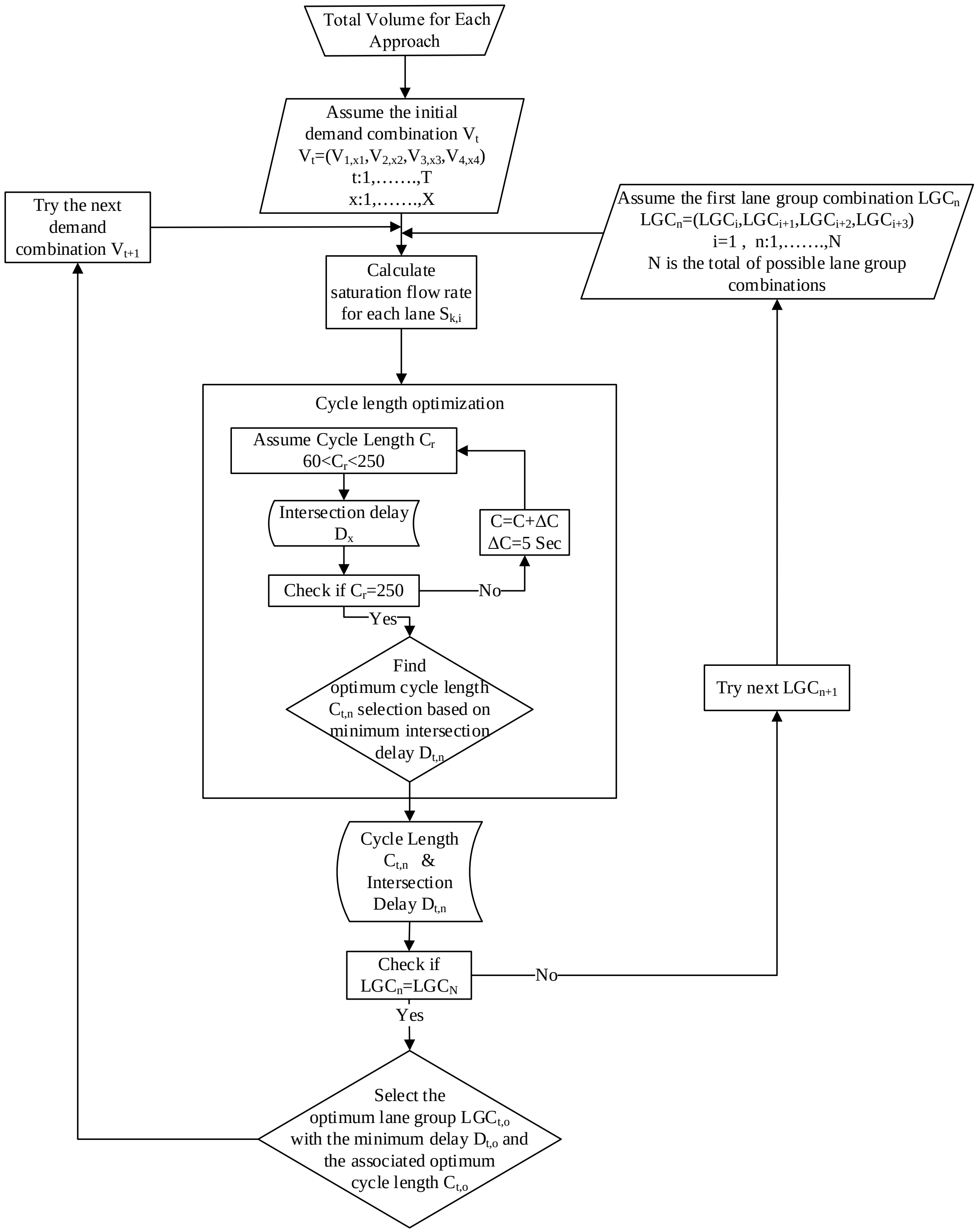


Figure 3.3: Flowchart for model formulation

3.3 Model Development

This research is performed on isolated intersection with N_T approaches. It is assumed that the number of approaching lanes N_i is not greater than the number of exit lanes. A binary function is defined to identify the permitted movements "*j*" from lane "*k*" at approach "*i*" as follow:

$$\alpha_{i,k,j} = \begin{cases} 0 & \text{movement } j \text{ using lane } k \text{ at approach } i \text{ is not allowed} \\ 1 & \text{otherwise} \end{cases}$$

where:

i: Intersection approach number, where *i* as shown in Figure 3.2.

k: Approach lane number, k=1, 2, 3….. N^i_A (numbered from median side to curbside lanes)

j: Turning movements at the intersection, j=1, 2, 3 respectively representing LT, TH , and RT movement.

If turning movement *j* is allowed at lane k+1, then for safety reasons all movements to j+1,2……..N_T should be prohibited at lane *k* to eliminate any potential conflicts in approach *i*. For instance if lane k+1 is assigned to LT movement then lane *k* cannot be assigned to TH or RT traffic movement.

To develop the model different component is required as shown in Figure 3.4. Starting with demand variation that explains how the traffic movement is fluctuated within the intersection. Secondly, all possible LGCs will be shown based on the intersection geometry. Thirdly, the equations of the saturation flow rate will be shown. Fourthly, the cycle length optimization will be discussed. Finally, the delay estimation method will be explained. Each component will be expressed in the following sub-section.

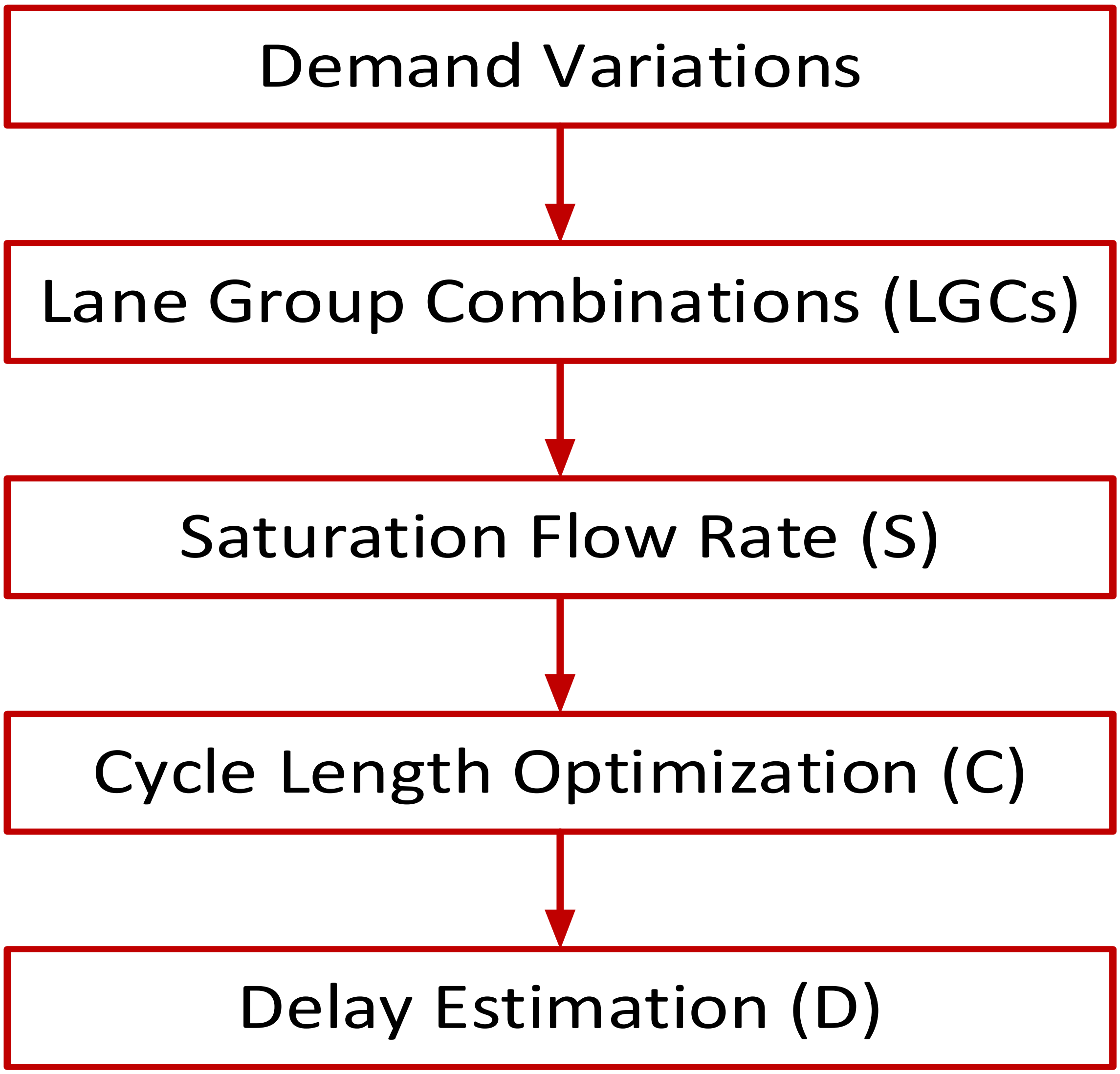


Figure 3.4: Model components flow chart

3.3.1 Demand Variations

The concept of movement groups which is used in this research is also established to facilitate data entry. A separate movement group is established for each turn movement with one or more exclusive turn lanes with the through movement (inclusive of any turn movements that share a lane) [15].

The demand variation for any approach is considered by changing the LT, TH and RT demands independently by an increment, the overall approach demand is fixed. The demand variation at any approach of the intersection, where DLG is applied, is defined based on the following logic:

$$\begin{aligned}
&\textit{Total approach demand } V_i = V_{LTi} + V_{THi} + V_{RTi} \\
&\textit{For each } V_{LTi} = \alpha V_i \textit{ to } 0.90V_i \\
&\textit{For each } V_{THi} = \beta V_i \textit{ to } (0.95V_i - V_{LTi}) \\
&V_{RTi} = V_i - V_{LTi} - V_{THi}
\end{aligned} \tag{1}$$

where:

$\boldsymbol{V_i}$: Total traffic volume for approach i (veh/h)

$\boldsymbol{V_{LTi}}$: Left-turning traffic volume for approach i (veh/h)

$\boldsymbol{V_{THi}}$: Through traffic volume for approach i (veh/h)

$\boldsymbol{V_{RTi}}$: Right-turning traffic volume for approach i (veh/h)

α, β: Initial proportion of LT and TH movement respectively (%)

For the purpose of this study, LT and TH vehicle demands at the approach, where DLG is applied, are assumed to change by an increment of 5% of the total approach demand.

Based on Equation (1) and the used increment, different demand combinations can be identified for each total approach demand volume (V_i).

3.3.2 Saturation Flow Rate

For each demand combination, the saturation flow ratio will be estimated for each lane in the intersection. For turning lanes, the saturation flow ratio is defined according to Equation (2) [43]:

$$S_{i,k} = \frac{\bar{\mathrm{S}}_{\ i,k}}{1 + 1.5 \sum_{j=1}^{j=3} (\frac{f_{\,i,k,j}}{r_{i,k,j}})} \tag{2}$$

where:

$\mathbf{S_{i,k}}$: Saturation flow rate of lane *k* in arm *i*.

$\bar{\mathbf{S}}_{\ \mathbf{i,k}}$: Saturation flow rate for straight movement (assumed to be 1900veh/h)

$\mathbf{r}_{\ \mathbf{i,j,k}}$: Turning radius for movement *j* (= ∞ for straight-ahead movement).

$\boldsymbol{f}_{\boldsymbol{i,j,k}}$: Flow factor (calculated from equation (5)).

The saturation flow ratio for a given lane *k* at approach *i* is defined as in equation (3):

$$\mathrm{y}_{i,k} = \frac{\sum_{j=1}^{j=3} V_{\ i,k,j}}{\mathrm{S}_{i,k}} \tag{3}$$

3.3.3 Flow Factor

The flow factor $f_{i,j,k}$ is defined as the proportion of traffic movement *j* from approach *i* via lane *k*. For all lanes that are not shared between two more movements, the flow factor $f_{i,k,j}$ equal to 0 or 1. However, if there are shared lanes, the flow factor is estimated following the assumption of equal saturation flow ratio for shared lanes and the adjacent traffic lanes. The flow factor for movement *j* at lane *k* of approach *i* from the traffic at lane *k* is defined as Equation (5):

$$f_{i,k,j} = \frac{\boldsymbol{V}_{\boldsymbol{i},k,j}}{\sum_{\boldsymbol{j=1}}^{\boldsymbol{j=3}} \boldsymbol{V}_{\boldsymbol{i},k,j}} \tag{5}$$

where:

$\boldsymbol{V}_{\mathbf{i,j,k}}$: Flow rate of movement *j* via lane *k* at approach *i* (veh/h).

3.3.4 Lane Group Combinations (LGC)

A lane is defined as a division of a road marked off with painted lines and intended to separate single lines of traffic according to speed or direction. Lane group is a set of lanes established at an intersection approach for separate capacity and level-of-service analysis. Lane groups are defined by one or more lanes that accommodate traffic with a common stop-line and a capacity that is shared by all vehicles. In general, a separate lane group is established for each lane (or combination of adjacent lanes) that exclusively serves one movement or two-shared movements or more [15].

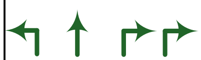
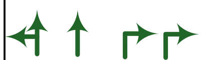
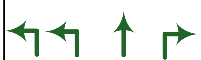
At each approach, traffic lanes can be divided into several lane groups based on the allowed movement/s per lane. A Lane Group Combination (LGC_i) represents the defined lane groups at approach *i*. At each approach, there are several possible LGCs based on number of available lanes.

To identify the possible LGCs in this study several assumption are made:

1. All movements have to be permitted in any LGC_i. This means that any LGC_i which prohibit a traffic movement (LT, TH, and RT) is neglected.
2. It is allowed to have shared lanes only at the far-left lane (median lane) and for the outer lane (shoulder lane).

Following previous assumptions and the used analysis site layout (Figure 3.2), the possible LGCs for the NA/SA (3-lanes) and WA/EA (4-lanes) approaches are shown in Table 3.1 .Ten LGCs are considered for the WA and EA while six LGCs are considered for the NA and EA. It is important to mention that the developed model can adopt any number of LGCs for approaches with different number of lane.

Table 3.1: Possible LGCs

WA & EA (4-lanes)		NA & SA (3-lanes)	
$LGC_{n,i}$ Where i=1,3 n=1-10	Assigned movement/s per lane	$LGC_{n,i}$ Where i=2,4 n=1-6	Assigned movement/s per lane
LGC1,i		LGC1,i	
LGC2,i		LGC2,i	
LGC3,i		LGC3,i	
LGC4,i		LGC4,i	
LGC5,i		LGC5,i	
LGC6,i		LGC6,i	
LGC7,i			
LGC8,i			
LGC9,i			
LGC10,i			

3.3.5 Cycle Length Optimization

To identify the optimum cycle length, an algorithm that is based on iterative process is developed. A minimum cycle length of 60 sec is adopted followings HCM 2010 recommendation as the acceptable cycle length to serve pedestrians meanwhile no limitations is proposed for the maximum cycle length which is selected by the local jurisdiction.

For the purpose of this study, a maximum cycle length of 250 sec is used which is similar to the adopted maximum cycle length by local authorities in Khobar and Dammam areas.

Using an increment of 5 sec, average intersection delay is estimated for all cycles between 60 sec and 250 sec. The cycle length that results in the minimum average intersection delay for a specific demand combination using a specific lane group is selected as the optimized cycle length for the demand combination and the LGC under consideration, these steps of cycle length optimization is shown in Figure 3.5.

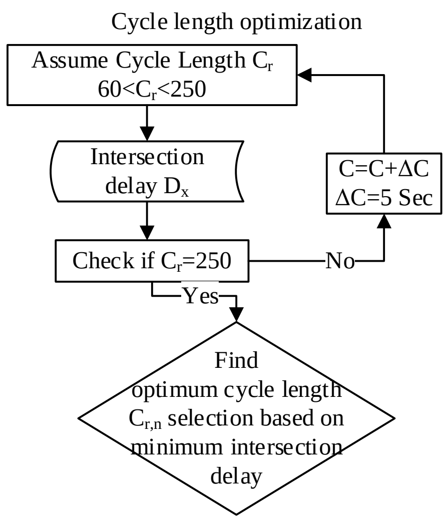


Figure 3.5: Cycle length optimization flow chart

Based on the assumed phasing plan (Figure 3.2); each phase will have both lost time (Table 3.2) and green split.

Table 3.2: Lost time assumptions [44]

Start-up lost time (l_1) = 2 sec/phase
Motorist use of yellow and all-red (e) = 2 sec/phase
Length of yellow change interval (y) = 3 sec
Length of all red clearance interval (ar)=1.5 sec
Total lost time for phase i (t_{Li})= 4.5 sec
Total lost time per cycle (L)= 18 sec

For each phase, the green split will be assigned based on multiplying the percentage of the critical v/s ratio from the total critical v/s for all approaches by the effective green time. Total effective green time is equal to the cycle length without the total lost time.

3.3.6 Delay Estimation

The equations of Highway Capacity Manual (HCM) 2010 were used to calculate the total intersection delay per vehicle. The average control delay per vehicle for a given lane group is given by Equation 5 [15].

$$d_{i,k} = d_{1,i,k}(PF) + d_{2,i,k} + d_{3,i,k} \tag{5}$$

where:

$d_{i,k}$: Control delay per vehicle (sec)

$d_{1,i,k}$: Uniform control delay assuming uniform arrivals (sec)

PF: Uniform delay progression adjustment factor, which accounts for effects of signal progression, assumed to be 1.

$d_{2,i,k}$: Incremental delay which is average delay per vehicle due to random arrivals (sec)

$d_{3,i,k}$: Average delay per vehicle due to initial queue at start of analysis time period (sec)

The average delay due to uniform arrivals is estimated according to HCM 2010 using Equation (6):

$$d_{1,i,k} = \frac{0.5C(1-\frac{g_i}{C})^2}{1-[min\left(1, x_{i,k}\right).\frac{g_i}{C}]} \tag{6}$$

where:

C: Cycle length (sec)

g_i: Effective green time for lane group (sec)

$X_{i,k}$: Lane group volume-to-capacity ratio (v/c), where $c_{i,k} = \mathrm{S}_{\mathrm{i,k}} (\frac{\mathrm{g_i}}{\mathrm{C}})$

The incremental delay is estimated using HCM 2010 as following Equation (7):

$$d_{2,i,k} = 900T\left[\left(x_{i,k}-1\right)+\sqrt{\left(x_{i,k}-1\right)^2+\left(\frac{8k_f I X_{i,k}}{c_{i,k}T}\right)}\right] \tag{7}$$

where:

T: Duration of analysis period (h).

k_f: Incremental delay factor.

I: Upstream filtering/metering adjustment factor.

$C_{i,k}$: Lane group capacity (veh/h).

$X_{i,k}$: Lane group volume to capacity ratio (v/c), where $c_{i,k} = \mathrm{S}_{\mathrm{i,k}} \left(\frac{\mathrm{g_i}}{\mathrm{C}}\right)$.

The value of the upstream filtering-metering adjustment factor I is assumed as 1.0 since the analysis site is assumed to be isolated intersection. Also since a demand responsive signal control strategy is incorporated in the proposed model, the value of the incremental delay factor k_f is assumed 0.50 in the developed model since the control type is assumed as non-actuated signal (Table 3.3). However the model can be adjusted to consider actuated signals.

Table 3.3: k-values to account for controller type [15]

Unit Extension (s)	Degree of Saturation (X)					
	≤ 0.50	0.60	0.70	0.80	0.90	≥ 1.0
≤ 2.0	0.04	0.13	0.22	0.32	0.41	0.50
2.5	0.08	0.16	0.25	0.33	0.42	0.50
3.0	0.11	0.19	0.27	0.34	0.42	0.50
3.5	0.13	0.20	0.28	0.35	0.43	0.50
4.0	0.15	0.22	0.29	0.36	0.43	0.50
4.5	0.19	0.25	0.31	0.38	0.44	0.50
5.0[a]	0.23	0.28	0.34	0.39	0.45	0.50
Pretimed or nonactuated movement	0.50	0.50	0.50	0.50	0.50	0.50

In this study, it is assumed that there is no initial queue delay from the previous analysis period which means that $d_{3,i,k}$ is assumed to be 0.

The procedure for delay estimation yields the control delay per vehicle for each lane group. It is often desirable to aggregate these values to provide delay for an intersection approach and for the intersection as a whole. This aggregation is done by computing weighted averages, where the lane group delays are weighted by the adjusted flows in the lane groups.

The average delay of an approach i is computed by using Equation (8) which is summation of each lane group delay multiplied with its associated volume divided by the total approach volume.

$$d_i = \frac{\sum d_{i,k} V_{i,k}}{\sum V_{i,k}} \tag{8}$$

where:

d_i: Delay for approach i (s/veh).

$d_{i,k}$: Delay for lane k in approach i (s/veh).

$V_{i,k}$: Adjusted flow for lane group for lane k in approach i (veh/h).

The average intersection delay is calculated using Equation (9) that is summation of multiplying each approach delay with its associated volume and divide by the total intersection traffic volume.

$$D_a = \frac{\sum d_i V_i}{\sum V_i} \tag{9}$$

where:

D_a: Delay per vehicle for intersection (s/veh).

d_i: Delay for approach i (s/veh).

V_i: Flow for approach i (veh/h).

The LGC which results in the minimum average intersection delay D_a for a specific demand combination will be selected as the optimal lane group combinations $LGC_{o,n}$ for that demand combination.

CHAPTER FOUR

4 RESULTS AND DISCUSSION

This chapter presents the results and discussions of the research findings. The results and discussions have been divided into two sections. First part compares the performance of the proposed strategy (DLG) with the FLG using different MOEs. The final part addresses the validation of the benefits of the model in real world. Empirical traffic data at a signalized intersection is used to assess the performance of DLG technique over FLG.

4.1 Effectiveness of DLG over FLG

A comparison analysis between DLG and FLG is made to identify the effectiveness of applying the new technique (DLG) over the existing one (FLG). This comparison was simplified since the required time for full run will take almost two months. Also, the limitation of the Excel sheet (1.048 Million rows) restricts the full run of the developed model.

For the simplification of the comparison, two approaches WA/EA as shown in Figure 4.1 in the selected intersection were used to perform the analysis. To represent the actual traffic conditions at the intersection in the optimization model, a traffic count study was conducted at the targeted intersection during the morning peak, afternoon peak and evening peak as shown in Table 4.1.

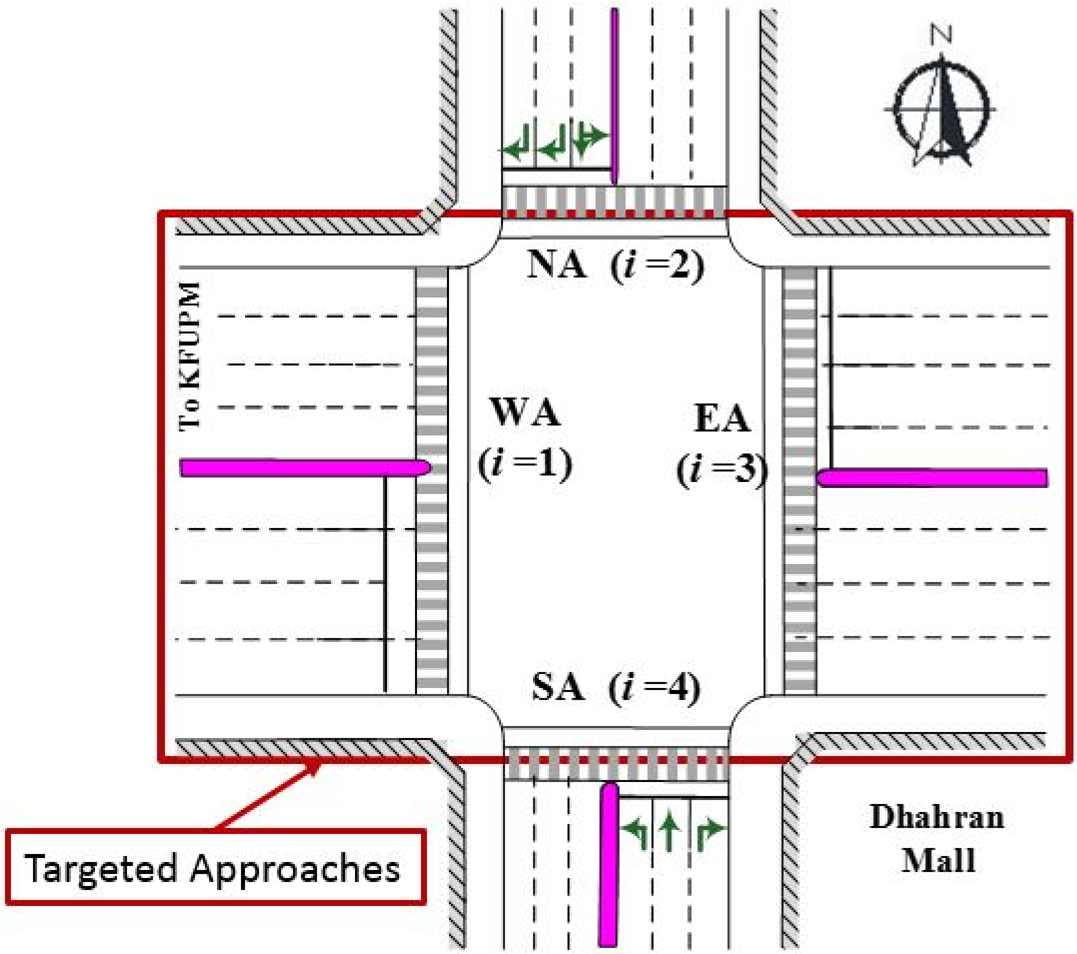


Figure 4.1: Targeted approaches

It can be observed from Table 4.1 that the intersection is suffering from a high traffic demand in the WA-EA directions during the peak hours. Hence, these two approaches were chosen to be the targeted approaches of this analysis with 1500 veh/h traffic volume. Traffic demand of RT movement for these approaches assumed to be 10.0% of the total approach volume. It was assumed that the other two approaches have a fixed lane movement groups as existing condition as shown in Figure 4.2. The AM peak traffic volumes, which can be considered as the lowest volumes comparing with the other peaks' traffic volumes, were assigned to the other approaches to reduce the effect of these approaches on the determination of optimized cycle length.

Table 4.1: Traffic volume counts by movement (veh/h)

		West approach			**North approach**			**East approach**			**South approach**			**Cycle length**
Period		V_L	V_{TH}	V_R	V_L	V_{TH}	V_R	V_L	V_{TH}	V_R	V_L	V_{TH}	V_R	
Morning peak (6-8)AM		850	529	9	160	36	278	11	1361	90	42	20	25	
Afternoon peak (11.30 AM - 1.30PM)		841	669	118	414	59	157	470	900	42	352	107	152	**160 sec Fixed for whole day**
Evening peak (5-8)PM		703	1125	371	329	90	250	661	1105	155	270	78	177	
Signal parameters (sec)	g_i	45	45	45	25	25	25	50	50	50	20	20	20	
	y_i	3	3	3	3	3	3	3	3	3	3	3	3	
	AR_i	2	2	2	2	2	2	2	2	2	2	2	2	

The developed model was used two times, one for FLG and the other for DLG, to conduct this analysis. Same volume combinations are assigned for both cases; and same intersection layout (Figure 4.2) for the DLG and (Figure 4.3) for FLG. In FLG, it is assumed that the RT movement is shared with TH movement. Based on this assumption Right Turn on Red (RTOR) is not considered in this situation. Furthermore, it implies no channelization for the RT movement. In this analysis $\alpha$and β are considered 5%.

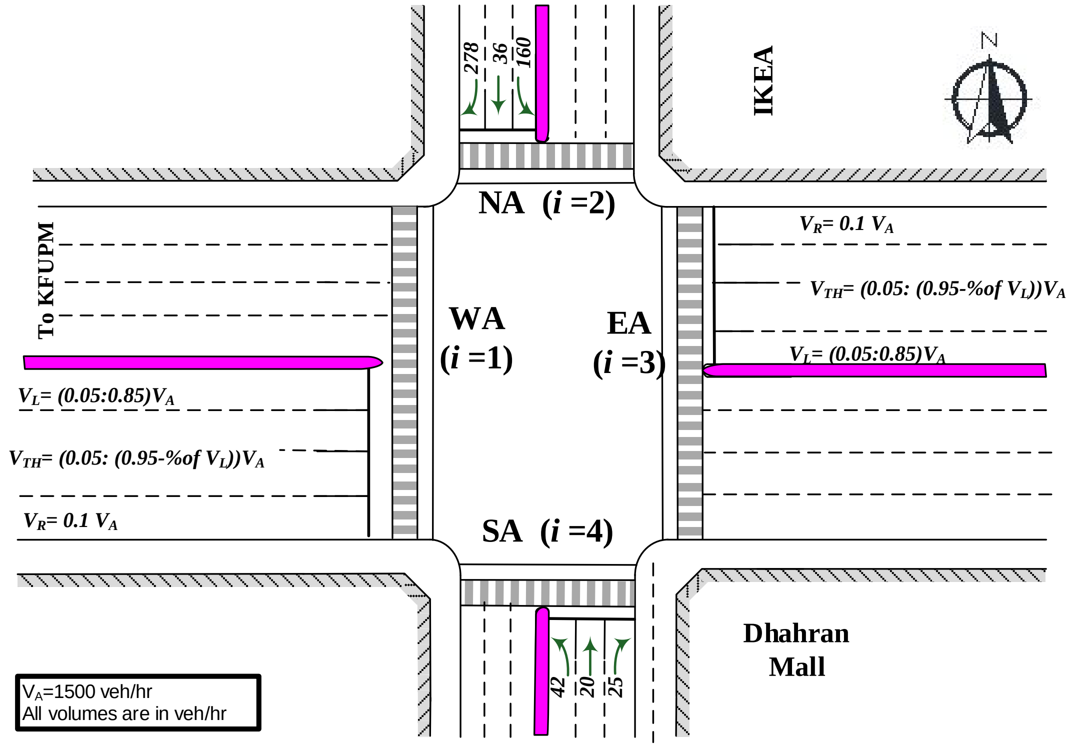


Figure 4.2: Two approaches condition for DLG

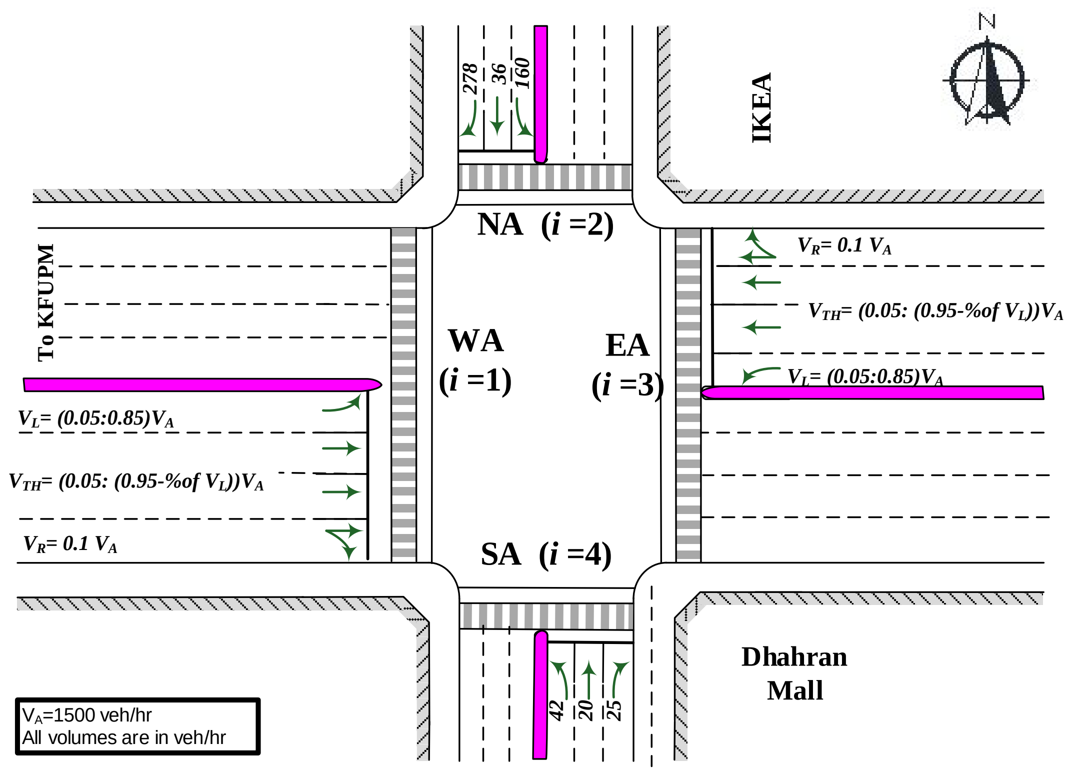


Figure 4.3: Two approaches condition for FLG

It is important to mention that more than 60.0% of the LT movement is excluded from the analysis and it will be hatched in the tables. Since, in reality, the percentage or LT movement will not exceed 60.0% of the total approach volume.

4.1.1 DLG Two Approaches Analysis

The output for applying DLG on two approaches is shown in Table 4.2 for the average intersection delay in second per vehicle. It can be observed that the maximum delay (52.1) sec/veh happens when the percentage of LT volume is 60.0% of the total approach traffic volume in both WA and EA approaches. The minimum delay (41.0) sec happens when the percentage of LT volume is 20.0% of the total approach volume in both WA and EA approaches. Also, it is clearly shown that in most cases when the percentage of the LT volume increases the delay increases. The average is (44.5) sec/veh with a standard deviation of (2.5) sec/veh.

Table 4.2: Average intersection Delay using DLG (Sec/veh)

Average Intersection Delay (Sec/Veh)		% of V_L in EA approach																
		0.05	**0.10**	**0.15**	**0.20**	**0.25**	**0.30**	**0.35**	**0.40**	**0.45**	**0.50**	**0.55**	**0.60**	**0.65**	**0.70**	**0.75**	**0.80**	**0.85**
% of V_L in WA approach	**0.05**	41.57	41.77	41.31	41.48	41.52	44.17	45.70	43.56	42.52	42.65	43.82	46.29	46.93	43.43	43.68	44.62	46.45
	0.10	41.77	41.97	41.50	41.68	41.72	44.37	45.94	43.77	42.74	42.87	44.04	46.53	47.16	43.64	43.89	44.86	46.69
	0.15	41.31	41.50	41.03	41.11	41.20	43.89	45.31	43.20	42.08	42.19	43.45	45.86	46.62	43.16	43.20	44.12	45.94
	0.20	41.48	41.68	41.11	41.02	41.19	44.19	45.38	43.23	41.96	42.05	43.48	45.92	46.98	43.32	42.92	43.96	45.77
	0.25	41.52	41.72	41.20	41.19	41.32	44.17	45.48	43.34	42.15	42.26	43.59	46.03	46.93	43.37	43.20	44.17	46.01
	0.30	44.17	44.37	43.89	44.19	44.17	47.04	49.04	46.55	45.36	45.51	46.85	49.67	50.23	46.25	46.76	47.84	49.97
	0.35	45.70	45.94	45.31	45.38	45.48	49.04	50.69	48.01	46.54	46.67	48.30	51.38	52.51	47.92	47.87	49.08	51.33
	0.40	43.56	43.77	43.20	43.23	43.34	46.55	48.01	45.61	44.29	44.40	45.88	48.62	49.68	45.58	45.46	46.55	48.59
	0.45	42.52	42.74	42.08	41.96	42.15	45.36	46.54	44.29	43.01	43.10	44.55	47.14	48.28	44.36	44.01	45.07	46.96
	0.50	42.65	42.87	42.19	42.05	42.26	45.51	46.67	44.40	43.10	43.19	44.67	47.27	48.46	44.49	44.08	45.16	47.06
	0.55	43.82	44.04	43.45	43.48	43.59	46.85	48.30	45.88	44.55	44.67	46.16	48.93	49.98	45.85	45.75	46.84	48.88
	0.60	46.29	46.53	45.86	45.92	46.03	49.67	51.38	48.62	47.14	47.27	48.93	52.11	53.23	48.51	48.51	49.73	52.03
	0.65	46.93	47.16	46.62	46.98	46.93	50.23	52.51	49.68	48.28	48.46	49.98	53.23	53.90	49.28	49.91	51.11	53.53
	0.70	43.43	43.64	43.16	43.32	43.37	46.25	47.92	45.58	44.36	44.49	45.85	48.51	49.28	45.47	45.62	46.64	48.65
	0.75	43.68	43.89	43.20	42.92	43.20	46.76	47.87	45.46	44.01	44.08	45.75	48.51	49.91	45.62	44.93	46.15	48.11
	0.80	44.62	44.86	44.12	43.96	44.17	47.84	49.08	46.55	45.07	45.16	46.84	49.73	51.11	46.64	46.15	47.34	49.44
	0.85	46.45	46.69	45.94	45.77	46.01	49.97	51.33	48.59	46.96	47.06	48.88	52.03	53.53	48.65	48.11	49.44	51.73

Regarding the optimized cycle length using the new technique the result is shown in Table 4.3. From these results, it is observed that the cycle length falls within 80-100 sec. The average is (86.2) sec with a standard deviation of (5.6) sec.

Table 4.3: Optimized cycle length using DLG (Sec)

Optimized Cycle Length (sec)		% of V_L in EA approach																
		0.05	0.10	0.15	0.20	0.25	0.30	0.35	0.40	0.45	0.50	0.55	0.60	0.65	0.70	0.75	0.80	0.85
% of V_L in WA approach	0.05	80	80	80	80	80	90	90	85	80	80	85	90	95	85	85	85	90
	0.10	80	80	80	80	80	90	90	85	80	85	85	90	95	85	85	85	90
	0.15	80	80	80	80	80	85	90	85	80	80	85	90	90	85	80	85	90
	0.20	80	80	80	80	80	85	85	85	80	80	85	90	90	85	80	85	85
	0.25	80	80	80	80	80	85	90	85	80	80	85	90	90	85	80	85	90
	0.30	90	90	85	85	85	95	95	90	90	90	90	100	100	90	90	95	95
	0.35	90	90	90	85	90	95	100	95	90	90	95	100	105	95	90	95	100
	0.40	85	85	85	85	85	90	95	90	85	85	90	95	100	90	85	90	95
	0.45	80	80	80	80	80	90	90	85	80	80	85	90	95	85	85	85	90
	0.50	80	85	80	80	80	90	90	85	80	80	85	90	95	85	85	85	90
	0.55	85	85	85	85	85	90	95	90	85	85	90	95	100	90	85	90	95
	0.60	90	90	90	90	90	100	100	95	90	90	95	100	105	95	95	95	100
	0.65	95	95	90	90	90	100	105	100	95	95	100	105	110	95	95	100	105
	0.70	85	85	85	85	85	90	95	90	85	85	90	95	95	90	85	90	95
	0.75	85	85	80	80	80	90	90	85	85	85	85	95	95	85	85	85	90
	0.80	85	85	85	85	85	95	95	90	85	85	90	95	100	90	85	90	95
	0.85	90	90	90	85	90	95	100	95	90	90	95	100	105	95	90	95	100

As shown in the previous tables, DLG gives a stable operation in terms of intersection delay along with optimized cycle length.

The last result for this analysis is the optimum LGC which is shown in Table 4.4, these numbers are identified in Table 4.5, from this table it can be concluded that when the percentage of LT movement increases the required LGC is changing to the accommodate the traffic movement demand. It can be observed that when the percentage of LT movement in the traffic demand is within the range of (5.0-25.0%) of the total approach volume it requires LGC_6. This LGC_6 consists of shared in both LT and RT movement since this percentage is very low, the heavy demand is in the through movement. For the range of (25.0-30.0%) the LGC_8 is assigned. No shared lane in the left lane. Since the program is checking the volume-to-capacity ratio in order to identify if there is a necessity of shared lanes in both left and/or right. For the next range (30.0-65.0%) the assigned is LGC_5. This implies that two exclusive LT lanes are required since the LT volume is increasing it needs an additional lane. For the last range (65.0-85.0%) this lead to a heavy demand on the LT movement. It will require additional lanes for this movement which is clearly observed in this situation since the resulted is LGC_{10}.

Table 4.4: Optimum lane group using DLG ($LGC_{n,1}$, $LGC_{n,3}$)

Optimum Lane Group		% of V_L in EA approach			
		0.05-0.20	**0.25-0.30**	**0.35-0.60**	**0.65-0.85**
% of V_L in WA approach	**0.05-0.20**	6,6	6,8	6,5	6,10
	0.25-0.30	8,6	8,8	8,5	8,10
	0.35-0.60	5,6	5,8	5,5	5,10
	0.65-0.85	10,6	10,8	10,5	10,10

Table 4.5: Lane group combinations for four lanes

WA & EA (4-lanes)	
$LGC_{n,i}$ Where i=1,3 n=1-10	Assigned movement/s per lane
LGC1,i	↰ ↑ ↑ ↱
LGC2,i	↰ ↑ ↱ ↱
LGC3,i	↰↑ ↑ ↱ ↱
LGC4,i	↰ ↰ ↑ ↱
LGC5,i	↰ ↰ ↑ ↑↱
LGC6,i	↰↑ ↑ ↑ ↑↱
LGC7,i	↰↑ ↑ ↑ ↱
LGC8,i	↰ ↑ ↑ ↑↱
LGC9,i	↰↑ ↱ ↱ ↱
LGC10,i	↰ ↰ ↰ ↑↱

4.1.2 FLG Two Approaches Analysis

The output for applying FLG on two approaches is shown Table 4.6 for the average intersection delay in second per vehicle. It can be observed that the delay is more than (150.0) sec/veh happens when the percentage of LT volume is more than 50.0% of the total approach traffic volume in both WA and EA approaches. The minimum intersection delay (41.32 sec/veh) happens when the percentage of LT volume is 25.0% of the total approach volume in both WA and EA approaches. Also, it is clearly shown that in most cases when the percentage of the LT volume increases the delay increases. The average is 112.1 sec/veh with a standard deviation of 84.4 sec/veh.

Table 4.6: Average intersection Delay using FLG (Sec/veh)

Average Intersection Delay (Sec/Veh)		% of V_L in EA approach																
		0.05	**0.10**	**0.15**	**0.20**	**0.25**	**0.30**	**0.35**	**0.40**	**0.45**	**0.50**	**0.55**	**0.60**	**0.65**	**0.70**	**0.75**	**0.80**	**0.85**
% of V_L in WA approach	**0.05**	52.59	50.15	48.10	46.62	46.33	50.37	58.04	69.22	85.34	108.92	143.09	189.93	249.03	316.03	388.74	466.22	548.01
	0.10	50.15	47.91	46.00	44.63	44.37	48.07	55.07	65.20	79.60	100.48	130.96	173.12	228.07	291.89	361.84	436.75	516.06
	0.15	48.10	46.00	44.20	42.99	42.76	46.15	52.58	61.79	74.77	93.37	120.48	158.56	209.05	269.58	336.78	409.17	486.12
	0.20	46.62	44.63	42.99	41.77	41.61	44.91	50.88	59.42	71.30	88.10	112.40	147.05	193.67	251.68	317.63	389.98	467.92
	0.25	46.33	44.37	42.76	41.61	41.32	44.17	49.81	57.78	68.82	84.35	106.66	138.02	180.03	233.60	297.32	368.83	446.30
	0.30	50.37	48.07	46.15	44.91	44.17	47.04	53.56	63.22	77.03	96.88	124.32	159.87	202.05	249.87	302.84	360.77	423.88
	0.35	58.04	55.07	52.58	50.88	49.81	53.56	62.32	75.56	94.84	121.61	156.34	197.68	244.46	296.03	351.93	411.97	476.03
	0.40	69.22	65.20	61.79	59.42	57.78	63.22	75.56	94.33	120.64	154.90	195.77	242.24	293.46	349.00	408.64	472.24	539.75
	0.45	85.34	79.60	74.77	71.30	68.82	77.03	94.84	120.64	154.48	194.97	241.08	292.10	347.42	406.84	470.25	537.56	608.73
	0.50	108.92	100.48	93.37	88.10	84.35	96.88	121.61	154.90	194.97	240.74	291.41	346.56	405.80	469.03	536.19	607.24	682.13
	0.55	143.09	130.96	120.48	112.40	106.66	124.32	156.34	195.77	241.08	291.41	346.21	405.30	468.36	535.36	606.27	681.05	759.69
	0.60	189.93	173.12	158.56	147.05	138.02	159.87	197.68	242.24	292.10	346.56	405.30	468.25	535.10	605.87	680.54	759.09	841.52
	0.65	249.03	228.07	209.05	193.67	180.03	202.05	244.46	293.46	347.42	405.80	468.36	535.10	605.69	680.21	758.63	840.95	927.17
	0.70	316.03	291.89	269.58	251.68	233.60	249.87	296.03	349.00	406.84	469.03	535.36	605.87	680.21	758.47	840.67	926.78	1016.81
	0.75	388.74	361.84	336.78	317.63	297.32	302.84	351.93	408.64	470.25	536.19	606.27	680.54	758.63	840.67	926.65	1016.58	1110.44
	0.80	466.22	436.75	409.17	389.98	368.83	360.77	411.97	472.24	537.56	607.24	681.05	759.09	840.95	926.78	1016.58	1110.34	1208.04
	0.85	548.01	516.06	486.12	467.92	446.30	423.88	476.03	539.75	608.73	682.13	759.69	841.52	927.17	1016.81	1110.44	1208.04	1309.55

Regarding the optimized cycle length using the FLG, the result is shown in Table 4.7. From these results, it is observed that the cycle length falls within 80-250 Sec. The cycle length reaches the maximum allowed value (250 sec) when the LT volume for both WA and EA more than 45% of the total approach volume of the targeted approaches. The average is (162.5) sec with a standard deviation of (63.0) sec.

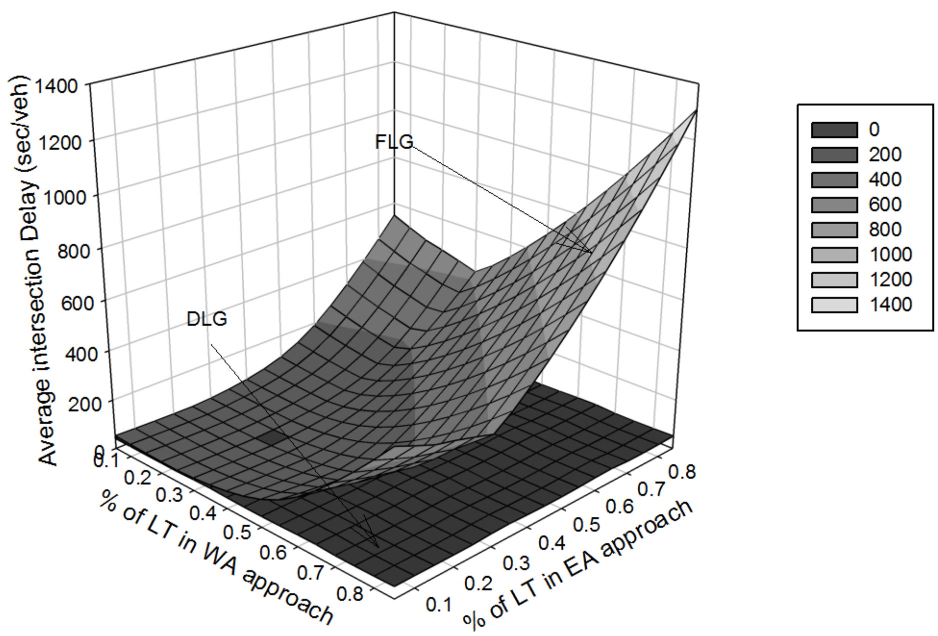
Table 4.7: Optimized cycle length using FLG

Optimized Cycle Length (sec)		% of V_L in EA approach																
		0.05	**0.10**	**0.15**	**0.20**	**0.25**	**0.30**	**0.35**	**0.40**	**0.45**	**0.50**	**0.55**	**0.60**	**0.65**	**0.70**	**0.75**	**0.80**	**0.85**
% of V_L in WA approach	**0.05**	100	95	90	90	90	100	115	135	160	195	235	250	250	250	250	250	250
	0.10	95	90	90	85	85	95	110	125	150	185	220	250	250	250	250	250	250
	0.15	90	90	85	80	80	90	105	120	145	175	210	245	250	250	250	250	250
	0.20	90	85	80	80	80	85	100	115	135	165	195	235	250	250	250	250	250
	0.25	90	85	80	80	80	85	100	115	130	160	190	220	250	250	250	250	250
	0.30	100	95	90	85	85	95	110	125	150	180	210	235	250	250	250	250	250
	0.35	115	110	105	100	100	110	125	150	180	210	235	250	250	250	250	250	250
	0.40	135	125	120	115	115	125	150	180	210	235	250	250	250	250	250	250	250
	0.45	160	150	145	135	130	150	180	210	235	250	250	250	250	250	250	250	250
	0.50	195	185	175	165	160	180	210	235	250	250	250	250	250	250	250	250	250
	0.55	235	220	210	195	190	210	235	250	250	250	250	250	250	250	250	250	250
	0.60	250	250	245	235	220	235	250	250	250	250	250	250	250	250	250	250	250
	0.65	250	250	250	250	250	250	250	250	250	250	250	250	250	250	250	250	250
	0.70	250	250	250	250	250	250	250	250	250	250	250	250	250	250	250	250	250
	0.75	250	250	250	250	250	250	250	250	250	250	250	250	250	250	250	250	250
	0.80	250	250	250	250	250	250	250	250	250	250	250	250	250	250	250	250	250
	0.85	250	250	250	250	250	250	250	250	250	250	250	250	250	250	250	250	250

As shown in the previous tables, FLG gives unstable operations in terms of intersection delay along with optimized cycle length.

4.1.3 Comparison Analysis (DLG vs FLG)

Applying DLG will utilize the available resources in the intersection. This utilization leads to a lower delay comparing with applying FLG strategy as shown clearly in Figure 4.4. This figure shows 3D surface representations of the estimated average intersection delays D_a for DLG and FLG at different demand combinations of the WA/EA approaches. A significant reduction in D_a is clearly observed after applying DLG. As the proportion of turning traffic at both approaches becomes larger, the reduction in D_a increases. This is attributed to the existing FLG combination at WA/EA which assigns only one lane exclusively to the LT movement in all cases of traffic demand for each approach even when the proportion of the LT movement is high. This also explains why the two surfaces are very close to each other when the proportion of the LT traffic is low since sufficient space is allocated to this movement by FLG, which is reasonable lane distribution according to the relative movement demand. However, when the proportion of the LT movement increased the D_a for the FLG is increasing since the assigned LT lane will not be sufficient to serve this movement. Furthermore, Figure 4.4 shows that the developed model stabilize intersection performance where the impacts of demand variations among different movements are significantly minimized.



Note1: The demands for NA and SA are the AM Peak hour demands shown in Table 4.1

Note2: For EA and WA 1500 veh/h is the total demand for each one.

Figure 4.4: Comparison between estimated minimum intersection delay D_a resulted from DLG and estimated D_a resulted from FLG with optimaized cycle length

A significant reduction in the intersection delay is obtained using DLG strategy as addressed in Table 4.8. The reduction varies from 0.0% for the cases that have similar LGC in DLG and FLG to 96.0% for the cases that have a huge LT traffic demand.

Table 4.8: % reduction in intersection delay

% of Reduction in Intersection Delay (sec)		% of V_L in EA approach																
		0.05	**0.10**	**0.15**	**0.20**	**0.25**	**0.30**	**0.35**	**0.40**	**0.45**	**0.50**	**0.55**	**0.60**	**0.65**	**0.70**	**0.75**	**0.80**	**0.85**
% of V_L in WA approach	**0.05**	20.96	16.71	14.11	11.02	10.37	12.31	21.25	37.08	50.17	60.84	69.37	75.63	81.16	86.26	88.76	90.43	91.52
	0.10	16.71	12.39	9.78	6.61	5.97	7.70	16.59	32.87	46.31	57.34	66.37	73.12	79.32	85.05	87.87	89.73	90.95
	0.15	14.11	9.78	7.17	4.37	3.66	4.90	13.82	30.09	43.72	54.81	63.94	71.08	77.70	83.99	87.17	89.22	90.55
	0.20	11.02	6.61	4.37	1.80	1.00	1.60	10.82	27.24	41.14	52.27	61.32	68.77	75.74	82.79	86.49	88.73	90.22
	0.25	10.37	5.97	3.66	1.00	0.00	0.00	8.68	24.99	38.75	49.91	59.13	66.65	73.93	81.44	85.47	88.02	89.69
	0.30	12.31	7.70	4.90	1.60	0.00	0.00	8.45	26.37	41.11	53.02	62.32	68.93	75.14	81.49	84.56	86.74	88.21
	0.35	21.25	16.59	13.82	10.82	8.68	8.45	18.67	36.47	50.93	61.62	69.11	74.01	78.52	83.81	86.40	88.09	89.22
	0.40	37.08	32.87	30.09	27.24	24.99	26.37	36.47	51.65	63.29	71.34	76.56	79.93	83.07	86.94	88.87	90.14	91.00
	0.45	50.17	46.31	43.72	41.14	38.75	41.11	50.93	63.29	72.16	77.89	81.52	83.86	86.10	89.10	90.64	91.62	92.29
	0.50	60.84	57.34	54.81	52.27	49.91	53.02	61.62	71.34	77.89	82.06	84.67	86.36	88.06	90.51	91.78	92.56	93.10
	0.55	69.37	66.37	63.94	61.32	59.13	62.32	69.11	76.56	81.52	84.67	86.67	87.93	89.33	91.44	92.45	93.12	93.57
	0.60	75.63	73.12	71.08	68.77	66.65	68.93	74.01	79.93	83.86	86.36	87.93	88.87	90.05	91.99	92.87	93.45	93.82
	0.65	81.16	79.32	77.70	75.74	73.93	75.14	78.52	83.07	86.10	88.06	89.33	90.05	91.10	92.76	93.42	93.92	94.23
	0.70	86.26	85.05	83.99	82.79	81.44	81.49	83.81	86.94	89.10	90.51	91.44	91.99	92.76	94.00	94.57	94.97	95.22
	0.75	88.76	87.87	87.17	86.49	85.47	84.56	86.40	88.87	90.64	91.78	92.45	92.87	93.42	94.57	95.15	95.46	95.67
	0.80	90.43	89.73	89.22	88.73	88.02	86.74	88.09	90.14	91.62	92.56	93.12	93.45	93.92	94.97	95.46	95.74	95.91
	0.85	91.52	90.95	90.55	90.22	89.69	88.21	89.22	91.00	92.29	93.10	93.57	93.82	94.23	95.22	95.67	95.91	96.05

In order to verify the significance of the benefits of DLG Table 4.9 provides a statistical comparison between average intersection delays $\overline{D_a}$ of DLG and that of FLG at different proportion of LT demand of the EA for all proportion of LT demand in WA for a total of 1500 veh/hr for each of the EA/WA approaches. It is clear that DLG always yield to significant reductions (at 95% confidence level) in D_a compared to FLG. This reduction increases as the proportion of turning increases where it can reach up to 96.1 reduction in D_a.

Table 4.9: Statistical comparison between DLG and FLG in terms of average intersection delay

Total traffic volume at WA (veh/h)	Proportion of V_R	Proportion of V_L in WA	Proportion of V_L in EA	Reduction in intersection delay $\%\Delta D_a$		Sample size	t-value
				$\overline{\%\Delta \boldsymbol{Da}}$	Range		
1500	0.1	0.05-0.85	0.05-0.20	45.4	1.0-91.5	68	-6.38
			0.25-0.40	47.1	0.0-91.0	68	-6.24
			0.45-0.60	75.1	38.8-93.8	68	-9.92
			0.65-0.85	89.6	73.9-96.1	85	-17.27

$$Note{:}\ \%\Delta D_a = \frac{D_a(FLG) - D_a\,(DLG)}{Da\,(FLG)} * 100\%$$

Table 4.10 : % reduction in optimized cycle length

% of Reduction in Cycle Length (Sec)		% of V_L in EA approach																
		0.05	**0.10**	**0.15**	**0.20**	**0.25**	**0.30**	**0.35**	**0.40**	**0.45**	**0.50**	**0.55**	**0.60**	**0.65**	**0.70**	**0.75**	**0.80**	**0.85**
% of V_L in WA approach	**0.05**	20.00	15.79	11.11	11.11	11.11	10.00	21.74	37.04	50.00	58.97	63.83	64.00	62.00	66.00	66.00	66.00	64.00
	0.10	15.79	11.11	11.11	5.88	5.88	5.26	18.18	32.00	46.67	54.05	61.36	64.00	62.00	66.00	66.00	66.00	64.00
	0.15	11.11	11.11	5.88	0.00	0.00	5.56	14.29	29.17	44.83	54.29	59.52	63.27	64.00	66.00	68.00	66.00	64.00
	0.20	11.11	5.88	0.00	0.00	0.00	0.00	15.00	26.09	40.74	51.52	56.41	61.70	64.00	66.00	68.00	66.00	66.00
	0.25	11.11	5.88	0.00	0.00	0.00	0.00	10.00	26.09	38.46	50.00	55.26	59.09	64.00	66.00	68.00	66.00	64.00
	0.30	10.00	5.26	5.56	0.00	0.00	0.00	13.64	28.00	40.00	50.00	57.14	57.45	60.00	64.00	64.00	62.00	62.00
	0.35	21.74	18.18	14.29	15.00	10.00	13.64	20.00	36.67	50.00	57.14	59.57	60.00	58.00	62.00	64.00	62.00	60.00
	0.40	37.04	32.00	29.17	26.09	26.09	28.00	36.67	50.00	59.52	63.83	64.00	62.00	60.00	64.00	66.00	64.00	62.00
	0.45	50.00	46.67	44.83	40.74	38.46	40.00	50.00	59.52	65.96	68.00	66.00	64.00	62.00	66.00	66.00	66.00	64.00
	0.50	58.97	54.05	54.29	51.52	50.00	50.00	57.14	63.83	68.00	68.00	66.00	64.00	62.00	66.00	66.00	66.00	64.00
	0.55	63.83	61.36	59.52	56.41	55.26	57.14	59.57	64.00	66.00	66.00	64.00	62.00	60.00	64.00	66.00	64.00	62.00
	0.60	64.00	64.00	63.27	61.70	59.09	57.45	60.00	62.00	64.00	64.00	62.00	60.00	58.00	62.00	62.00	62.00	60.00
	0.65	62.00	62.00	64.00	64.00	64.00	60.00	58.00	60.00	62.00	62.00	60.00	58.00	56.00	62.00	62.00	60.00	58.00
	0.70	66.00	66.00	66.00	66.00	66.00	64.00	62.00	64.00	66.00	66.00	64.00	62.00	62.00	64.00	66.00	64.00	62.00
	0.75	66.00	66.00	68.00	68.00	68.00	64.00	64.00	66.00	66.00	66.00	66.00	62.00	62.00	66.00	66.00	66.00	64.00
	0.80	66.00	66.00	66.00	66.00	66.00	62.00	62.00	64.00	66.00	66.00	64.00	62.00	60.00	64.00	66.00	64.00	62.00
	0.85	64.00	64.00	64.00	66.00	64.00	62.00	60.00	62.00	64.00	64.00	62.00	60.00	58.00	62.00	64.00	62.00	60.00

A significant reduction in the optimized cycle length is obtained using DLG strategy as addressed in Table 4.10. The reduction varies from 0.0% for the cases that have similar LGC in DLG and FLG to 68.0% for the cases that have a 45% LT traffic demand from the total approach volume.

Table 4.11: Statistical comparison between DLG and FLG in terms of optimized cycle length

Total traffic volume at WA (veh/h)	Proportion of V_R	Proportion of V_L in WA	Proportion of V_L in EA	Reduction in intersection delay $\%\Delta C_o$		Sample size	t-value
				$\overline{\%\Delta C_o}$	Range		
1500	0.1	0.05-0.85	0.05-0.20	37.9	0.0-68.0	68	-9.51
			0.25-0.40	39.4	0.0-68.0	68	-10.58
			0.45-0.60	60.1	38.5-68.0	68	-34.70
			0.65-0.85	63.5	56.0-68.0	85	-31.49

$$Note: \%\Delta C_o = \frac{C_o(FLG) - C_o\,(DLG)}{C_o\,(FLG)} * 100\%$$

In order to verify the significance of the benefits of DLG Table 4.11 provides a statistical comparison between optimized cycle length C_o of DLG and that of FLG at different proportion of LT demand of the EA for all proportion of LT demand in WA for a total of 1500 veh/hr for each of the EA/WA approaches. It is clear that DLG always yield to significant reductions (at 95% confidence level) in C_o compared to FLG.

4.2 Sensitivity and Validation of the Model

For this part of the study, it is important to say that due to security reasons, traffic count for 24 hrs cannot be performed. The traffic count (Table 4.1) is used in the new model. Traffic count was conducted during three different time periods within a typical day (Morning, afternoon, and evening). Using the traffic count for each period in the developed model resulted in different LGC and optimum cycle length as in Table 4.12 (refer to Table 3.1 for LGC). The results of DLG are compared with the existing FLG as shown in Figure 3.2. The result shows a significant reduction when DLG is applied at the intersection. For the morning period the cycle length of the FLG is 250.0 sec whereas it is 75.0 sec for DLG. The reduction of the cycle length is 70.0%. Moreover, the D_a is reduced 77.2% after applying the DLG. In the afternoon period the LGC is changed in three approaches. The LGCs assignment considers the variation in the traffic demand at any approach. This consideration enhances the performance of the signal at the intersection. For this period, the reduction in the cycle length is 52.0%. The D_a is also reduced by 81.7%. New LGC is assigned In the evening period. The new assignment reduces the cycle length by 2.1 % and the D_a by 4.1%. The lower reduction is observed since the intersection almost reaches its capacity. However, the DLG accommodates the best LGCs that are required for any variation in the traffic demand.

As a result, while the demand is changing between different times of the same day. The DLG model can handle this variation with assigning the best LGCs. The assignment of LGC_o will minimize the average intersection delay that lead to optimal cycle length.

Table 4.12: Results of peak demands using DLG and FLG

		West approach			North approach			East approach			South approach		
Period		V_L	V_{TH}	V_R	V_L	V_{TH}	V_R	V_L	V_{TH}	V_R	V_L	V_{TH}	V_R
Morning peak (6-8)AM		850	529	9	160	36	278	11	1361	90	42	20	25
FLG	LGC	8			4			8			4		
	C_o (Sec)	250											
	D (sec/veh)	165.35											
DLG	LGC	5			5			6			1		
	C_o (Sec)	75											
	D (sec/veh)	37.67											
Afternoon peak (11.30 AM -1.30PM)		841	669	118	414	59	157	470	900	42	352	107	152
FLG	LGC	8			4			8			4		
	C_o (Sec)	250											
	D (sec/veh)	383.47											
DLG	LGC	5			6			5			6		
	C_o (Sec)	120											
	D (sec/veh)	70.10											
Evening peak (5-8)PM		703	1125	371	329	90	250	661	1105	155	270	78	177
FLG	LGC	8			4			8			4		
	C_o (Sec)	235											
	D (sec/veh)	271.50											
DLG	LGC	8			1			8			6		
	C_o (Sec)	230											
	D (sec/veh)	260.40											

As it can be seen in the Table 4.12, (with Figure 4.5,Figure 4.6, and Figure 4.7) the DLG is sensitive for demand variation in different time period within the day. For the morning period, the DLG is assigned shared LT lane for EA but two exclusive LT lanes in the afternoon and only on for the evening period for the same direction. Also, for the other approaches as clearly shown the assigned LGCs are changed to accommodate with the traffic demand for any movement in any approach.

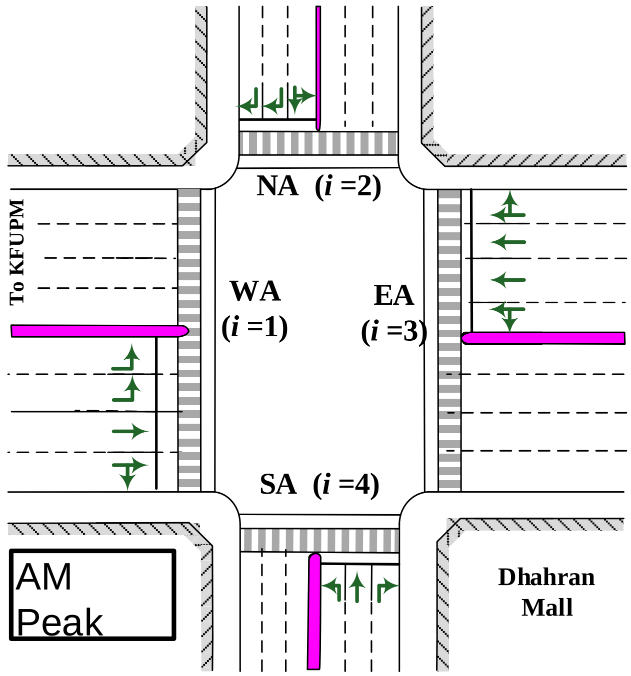


Figure 4.5: Assigned LGCs using DLG for Morning period.

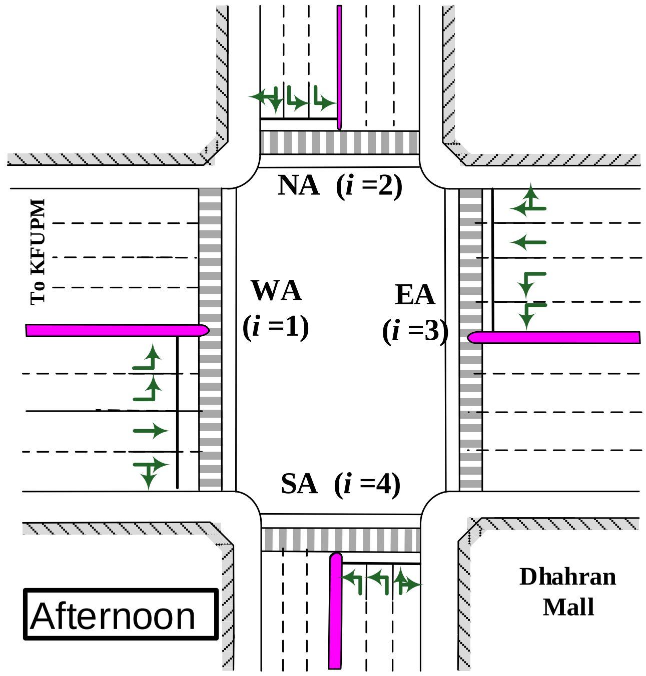


Figure 4.6: Assigned LGCs using DLG for Afternoon period.

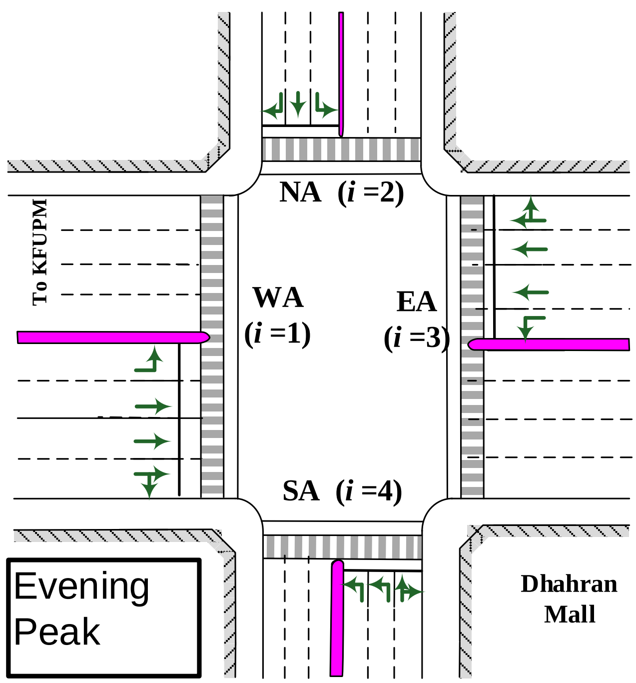


Figure 4.7: Assigned LGCs using DLG for Evening period.

CHAPTER FIVE

5 COMPARISON ANALYSIS

This chapter presents comparison between the developed model with well-known traffic software such as Synchro7 and HCS2000.

5.1 Comparison of the Outputs of the Model

In this section calculated delay and optimized cycle length are compared with those estimated by well-known traffic software (Synchro7) for optimizing traffic signal timing.

An intersection as shown in Figure 5.1 is built using Synchro (Figure 5.2) for the comparison analysis. Different volumes are used to compare the developed model. Figure 5.3 shows the input screen for the volumes and lane group combinations. Signal phasing for Synchro is addressed as shown in Figure 5.4 which shows four phasing plans as used in our model.

The next sub-section presents comparison between the developed model and Synchro in terms of cycle length and intersection delay. The differences between estimated delays and cycle lengths in the developed model and Synchro are statically tested using Excel software. Total sample size is 135. Different statistical coefficients are estimated to identify the relationship between the two models outputs.

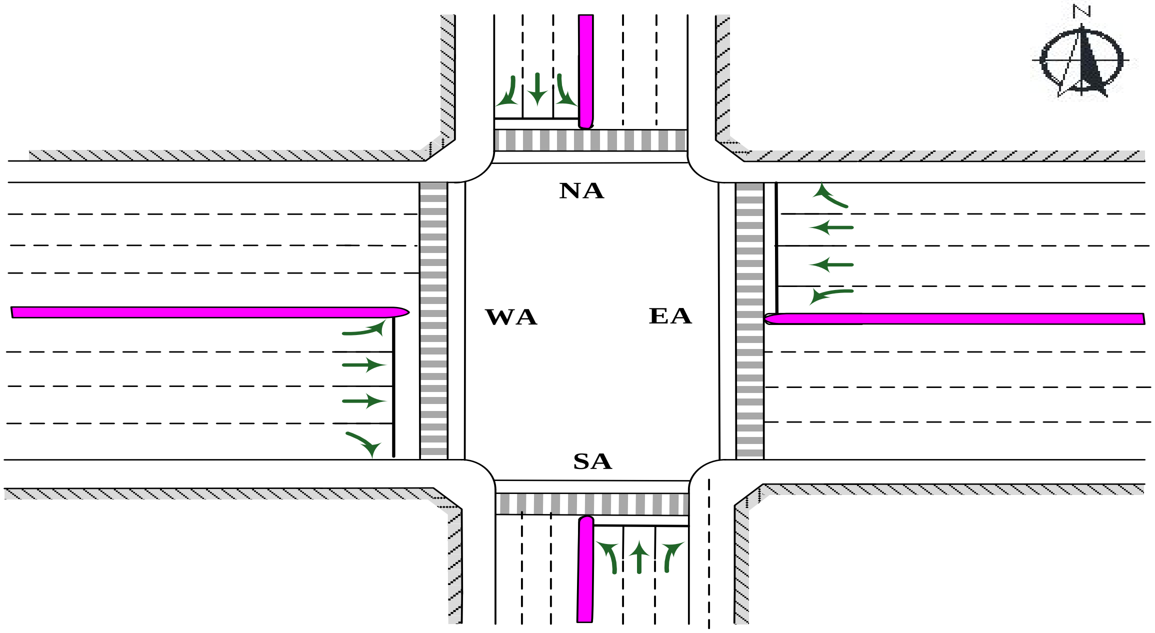


Figure 5.1: Intersection layout for output verifications

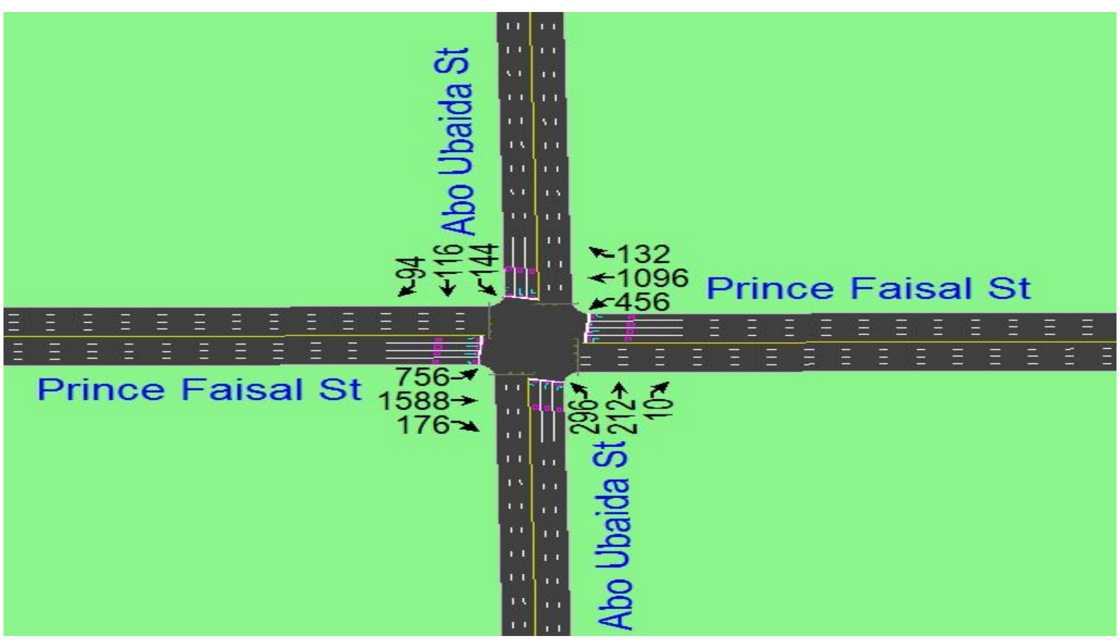


Figure 5.2: Intersection layout using Synchro software

LANE SETTINGS	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lanes and Sharing (#RL)	↰	↑↑	↱	↰	↑↑	↱	↰	↑	↱	↰	↑	↱
Traffic Volume (vph)	756	1588	176	456	1096	132	296	212	10	144	116	94
Street Name	Prince Faisal St			Prince Faisal St			Abo Ubaida St			Abo Ubaida St		
Link Distance (m)	—	1042.4	—	—	1167.4	—	—	983.1	—	—	792.7	—
Links Speed (km/h)	—	48	—	—	48	—	—	48	—	—	48	—
Set Arterial Name and Speed	—	EB	—	—	WB	—	—	NB	—	—	SB	—
Travel Time (s)	—	78.2	—	—	87.6	—	—	73.7	—	—	59.5	—
Ideal Satd. Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Lane Width (m)	3.7	3.7	3.7	3.7	3.7	3.7	3.7	3.7	3.7	3.7	3.7	3.7
Grade (%)	—	0	—	—	0	—	—	0	—	—	0	—
Area Type CBD	—	☐	—	—	☐	—	—	☐	—	—	☐	—
Storage Length (m)	0.0	—	0.0	0.0	—	0.0	0.0	—	0.0	0.0	—	0.0
Storage Lanes (#)	—	—	—	—	—	—	—	—	—	—	—	—
Right Turn Channelized	—	—	None	—	—	None	—	—	None	—	—	None
Curb Radius (m)	—	—	—	—	—	—	—	—	—	—	—	—
Add Lanes (#)	—	—	—	—	—	—	—	—	—	—	—	—
Lane Utilization Factor	1.00	0.95	1.00	1.00	0.95	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Right Turn Factor	1.000	1.000	0.850	1.000	1.000	0.850	1.000	1.000	0.850	1.000	1.000	0.850
Left Turn Factor (prot)	0.950	1.000	1.000	0.950	1.000	1.000	0.950	1.000	1.000	0.950	1.000	1.000
Saturated Flow Rate (prot)	1789	3579	1601	1789	3579	1601	1789	1883	1601	1789	1883	1601
Left Turn Factor (perm)	0.950	1.000	1.000	0.950	1.000	1.000	0.950	1.000	1.000	0.950	1.000	1.000
Right Ped Bike Factor	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Left Ped Factor	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Saturated Flow Rate (perm)	1789	3579	1601	1789	3579	1601	1789	1883	1601	1789	1883	1601
Right Turn on Red?	—	—	☐	—	—	☐	—	—	☐	—	—	☐
Saturated Flow Rate (RTOR)	0	0	0	0	0	0	0	0	0	0	0	0

Figure 5.3: Synchro inputs screen (volumes & LGC)

NODE SETTINGS	
Node #	1
Zone:	
X East (m):	2657.0
Y North (m):	1641.5
Z Elevation (m):	0.0
Description	
Control Type	Pretimed
Cycle Length (s):	185.0
Lock Timings:	☐
Optimize Cycle Length:	Optimize
Optimize Splits:	Optimize
Actuated Cycle(s):	185.0
Natural Cycle(s):	245.0
Max v/c Ratio:	1.14
Intersection Delay (s):	221.5
Intersection LOS:	F
ICU:	1.08
ICU LOS:	G
Offset (s) :	0.0
Referenced to:	Begin of Green
Reference Phase:	2 - WBTL
Master Intersection:	☐
Yield Point:	Single

TIMING SETTINGS	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR	PED	HOLD
Lanes and Sharing (#RL)	↰	↑↑	↱	↰	↑↑	↱	↰	↑	↱	↰	↑	↱	—	—
Traffic Volume (vph)	756	1588	176	456	1096	132	296	212	10	144	116	94	—	—
Turn Type	Split	—	Prot	Split	—	Prot	Split	—	Prot	Split	—	Prot	—	—
Protected Phases	1	1	1	2	2	2	3	3	3	4	4	4		
Permitted Phases													—	—
Detector Phases	1	1	1	2	2	2	3	3	3	4	4	4	—	—
Switch Phase	0	0	0	0	0	0	0	0	0	0	0	0	—	
Leading Detector (m)	15.2	15.2	15.2	15.2	15.2	15.2	15.2	15.2	15.2	15.2	15.2	15.2	—	—
Trailing Detector (m)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	—	—
Minimum Initial (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	—	—
Minimum Split (s)	21.0	21.0	21.0	21.0	21.0	21.0	21.0	21.0	21.0	21.0	21.0	21.0	—	—
Total Split (s)	76.0	76.0	76.0	57.0	57.0	57.0	31.0	31.0	31.0	21.0	21.0	21.0	—	—
Yellow Time (s)	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	—	—
All-Red Time (s)	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	—	—
Lost Time Adjust (s)	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	—	—
Lagging Phase?	☐	☐	☐	☑	☑	☑	☐	☐	☐	☑	☑	☑	—	—
Allow Lead/Lag Optimize?	☑	☑	☑	☑	☑	☑	☑	☑	☑	☑	☑	☑	—	—
Recall Mode	Max	Max	Max	Max	Max	Max	Max	Max	Max	Max	Max	Max	—	—
Actuated Effct. Green (s)	72.0	72.0	72.0	53.0	53.0	53.0	27.0	27.0	27.0	17.0	17.0	17.0	—	—
Actuated g/C Ratio	0.39	0.39	0.39	0.29	0.29	0.29	0.15	0.15	0.15	0.09	0.09	0.09	—	—
Volume to Capacity Ratio	1.09	1.14	0.28	0.89	1.07	0.29	1.13	0.77	0.04	0.88	0.67	0.64	—	—
Control Delay (s)	234.5	311.9	40.4	88.1	208.3	53.5	356.4	97.1	68.7	142.7	102.2	102.8	—	—
Queue Delay (s)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	—	—
Total Delay (s)	234.5	311.9	40.4	88.1	208.3	53.5	356.4	97.1	68.7	142.7	102.2	102.8	—	—
Level of Service	F	F	D	F	F	D	F	F	E	F	F	F	—	—
Approach Delay (s)	—	269.7	—	—	163.6	—	—	244.8	—	—	118.9	—	—	—

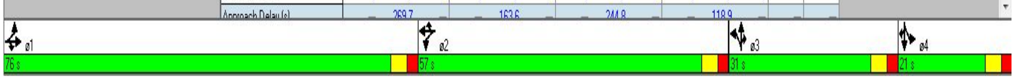


Figure 5.4: Synchro phasing plan

5.1.1 Cycle Length

The cycle lengths resulted from the developed model are compared with those resulted from the Synchro model. The main objective of this analysis is to compare the cycle length estimation model. It can be observed from Figure 5.5 that generally Synchro produces lower cycle lengths when compared with the developed model. However, it is not sensitive to the traffic volume change as the developed model. By assuming the cycle length of Synchro as the observed value and the cycle length of the developed model as the predicted value, the absolute percentage error range for the cycle length found to be from 0.0% to 72.0%. Mean Absolute Percentage Error (MAPE) measures the size of the error in percentage. It is equal 13.3% which is reasonable when compared with the range of data (60-250s). The coefficient of determination (R^2) found to be 93.2% which gives a good indication about the strength of the relationship between the results of both models.

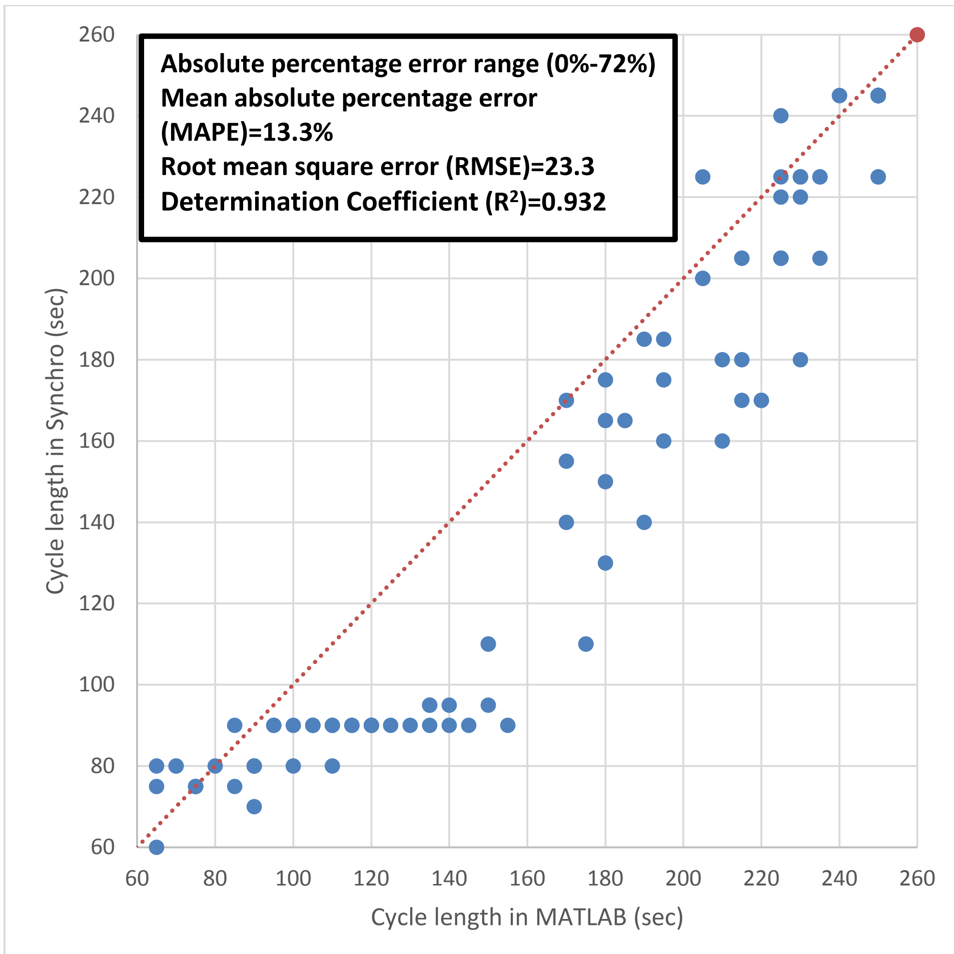


Figure 5.5: Cycle length comparison between MATLAB and Synchro

5.1.2 Intersection Delay

With Synchro7

A comparison is performed between the two models in terms of average intersection delay as shown in Figure 5.6. It is clear from the figure that almost all of the points are concentrated around the 45 degree line which means that the resulted average intersection delays from both models are very close. A good indication about the strength of the relationship between the results of both models based on R^2 value 99.4%

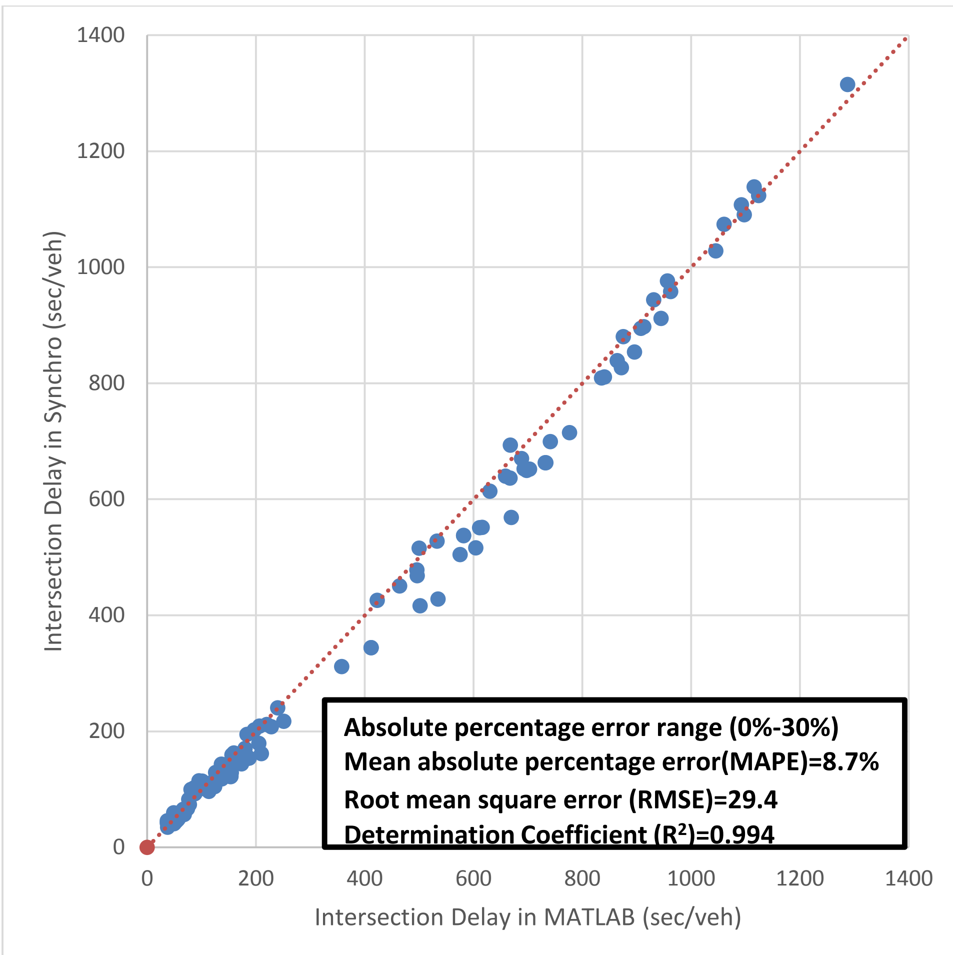


Figure 5.6: Intersection delay comparison between MATLAB and Synchro

With HCS 2000

Another comparison is performed between the developed model with HCS 2000 in terms of average intersection delay. The sample size for this comparison is 61. It is important to mention that intersection delay for the MATLAB model is developed using HCM 2010. HCS 2000 is based on HCM 2000. Fortunately, both manuals are using the same equation for calculating the intersection delay. Different input is required in HCS 2000 (volumes, factors like duration, upstream filtering and initial queue, phasing and saturation flow rate with associated factors). These inputs are shown in (Figure 5.7 to Figure 5.11). The average intersection delay of the HCS 2000 for one case is shown in Figure 5.12.

Figure 5.13 shows the results of both models. It is important to mention that intersection delay in this analysis is found at the same cycle length with similar green split for the developed model and HCS 2000. It is clear from the figure that the points are concentrated around the 45 degree line which means that the resulted average intersection delays from both models are very close. The absolute percentage error range for the average intersection delay is varied from 0.0% to 11.1%. MAPE equals to 2.4 % which is reasonable when compared with the range of data (40-1400 s/veh). The coefficient of determination (R^2) is 98.9% which gives a good indication about the strength of the relationship between the results of both models.

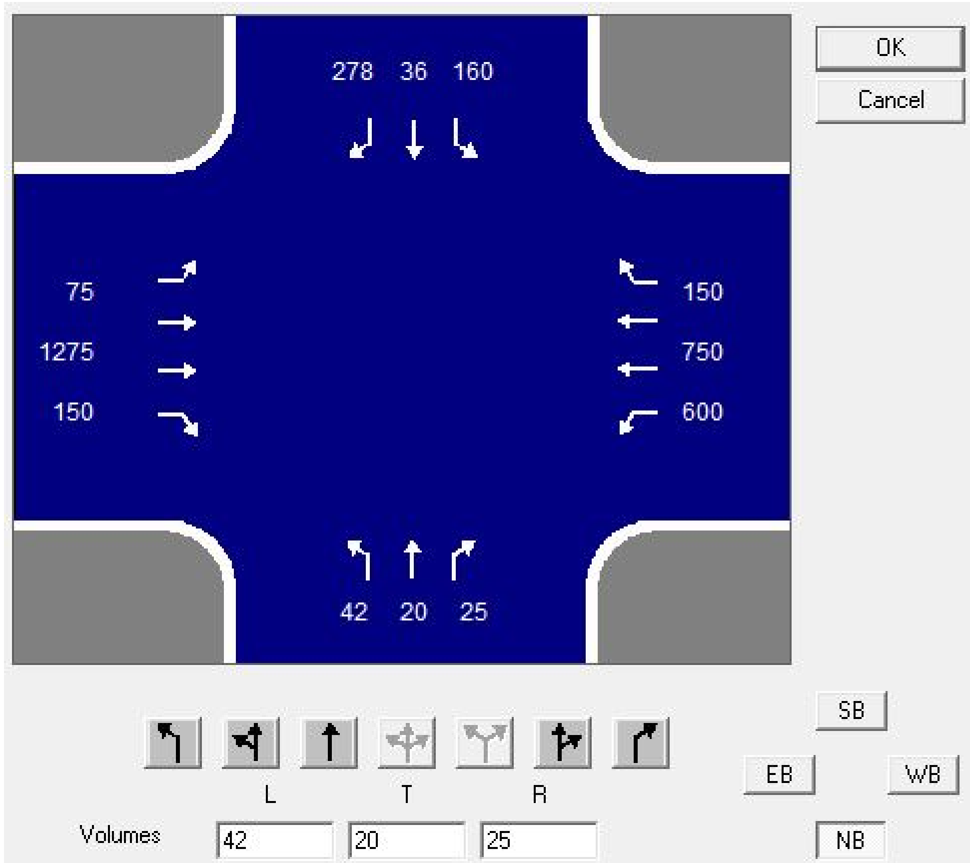
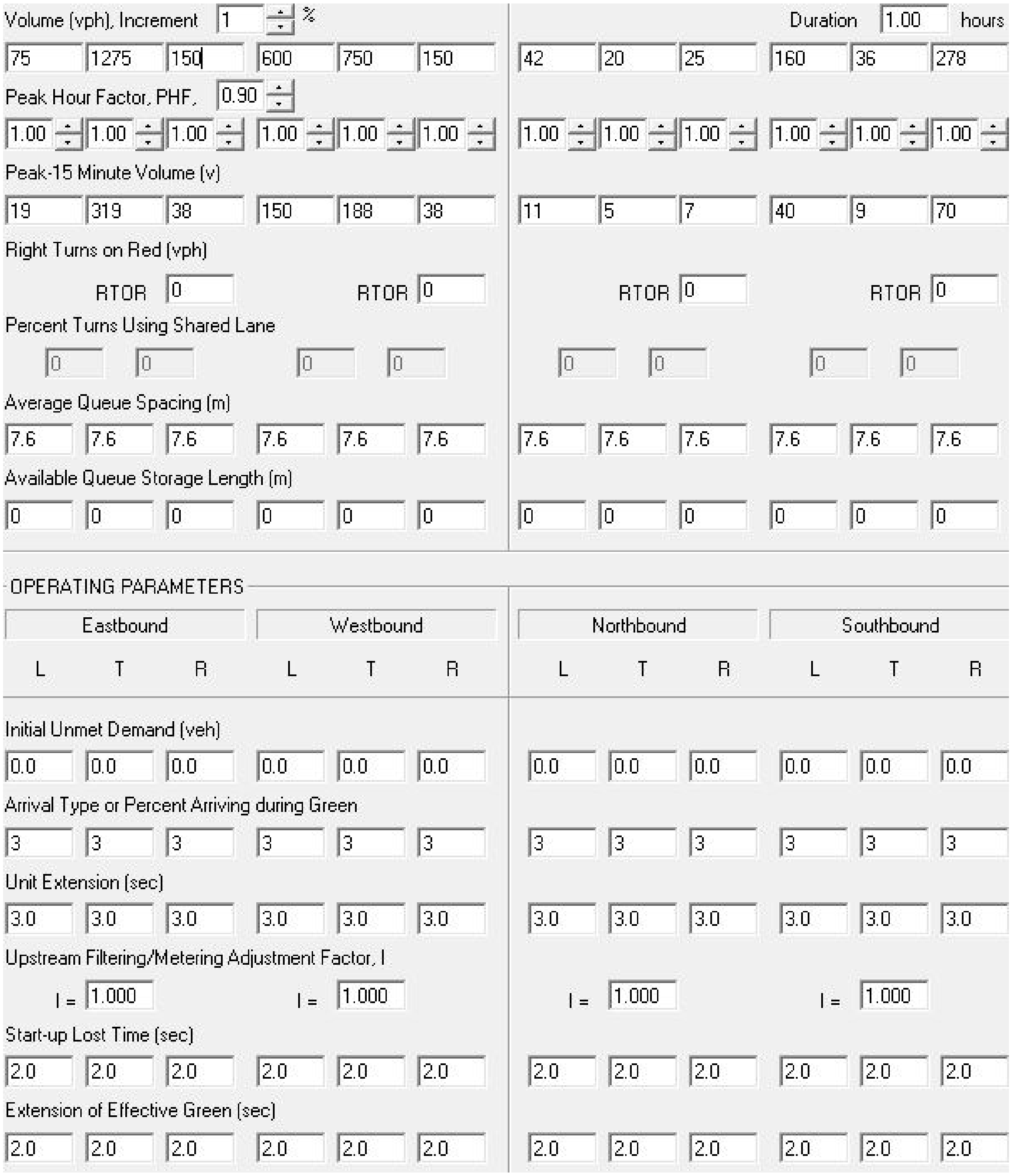


Figure 5.7: Volume input screen in HCS 2000



Volume (vph), Increment 1 % — Duration 1.00 hours

Volume (vph)	75	1275	150	600	750	150	42	20	25	160	36	278
Peak Hour Factor, PHF, 0.90	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Peak-15 Minute Volume (v)	19	319	38	150	188	38	11	5	7	40	9	70
Right Turns on Red (vph)		RTOR 0			RTOR 0			RTOR 0			RTOR 0	
Percent Turns Using Shared Lane	0		0	0		0	0		0	0		0
Average Queue Spacing (m)	7.6	7.6	7.6	7.6	7.6	7.6	7.6	7.6	7.6	7.6	7.6	7.6
Available Queue Storage Length (m)	0	0	0	0	0	0	0	0	0	0	0	0

OPERATING PARAMETERS

	Eastbound			Westbound			Northbound			Southbound		
	L	T	R	L	T	R	L	T	R	L	T	R
Initial Unmet Demand (veh)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Arrival Type or Percent Arriving during Green	3	3	3	3	3	3	3	3	3	3	3	3
Unit Extension (sec)	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
Upstream Filtering/Metering Adjustment Factor, I		I = 1.000			I = 1.000			I = 1.000			I = 1.000	
Start-up Lost Time (sec)	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
Extension of Effective Green (sec)	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0

Figure 5.8: Input factors in HCS 2000

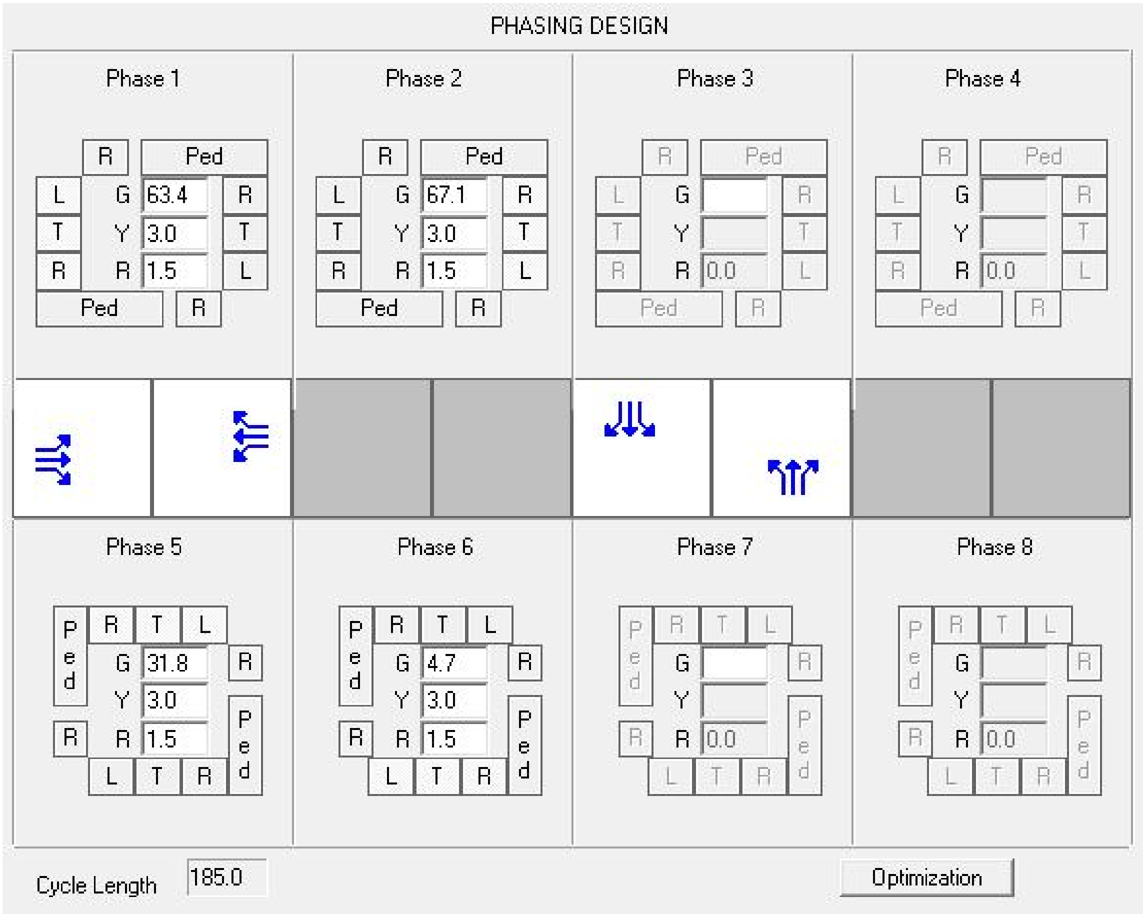


Figure 5.9: Phasing, Splits, Yellow and All red intervals in HCS 2000

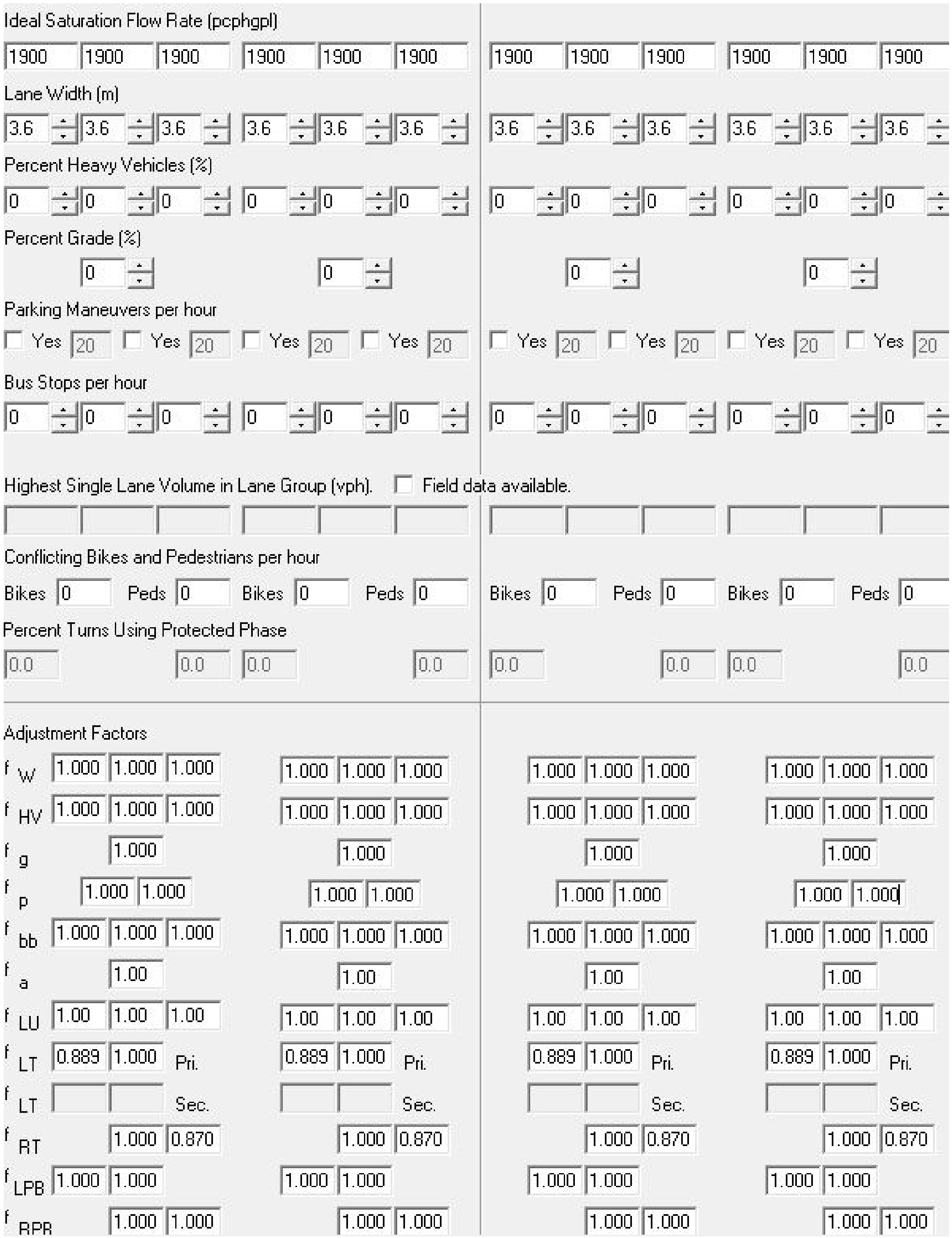


Figure 5.10: Ideal saturation flow ratio and associated factors in HCS 2000

Adjusted Saturation Flow Rate, vphg											
1689	3800	1653	1689	3800	1653	1689	1900	1653	1689	1900	1653

Figure 5.11: Adjusted Saturation flow rate in HCS 2000

RESULTS

	Eastbound			Westbound			Northbound			Southbound		
	L	T	R	L	T	R	L	T	R	L	T	R
Lane Group Adjusted Volume, (vph)	75	1275	150	600	750	150	42	20	25	160	36	278
Lane Group Capacity, (vph)	579	1185	566	613	1254	600	43	48	42	290	327	284
Lane Group v/c Ratio	0.13	1.08	0.27	0.98	0.60	0.25	0.98	0.42	0.60	0.55	0.11	0.98
Critical Lane Group		#		#			#					#
Lane Group Delay, (sec/veh)	42.3	216.4	45.1	113.6	50.1	42.3	341.3	114.9	147.6	77.6	65.3	164.6
Lane Group Level of Service	D	F	D	F	D	D	F	F	F	E	E	F
Final Unmet Demand, (v)	0.0	90.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Approach Delay, (sec/veh)		190.6			74.7			233.6			127.7	
Approach Level of Service		F			E			F			F	

Cycle Length 185.0 sec Intersection Delay 134.5 sec/veh Intersection LOS F

Figure 5.12: Average intersection delay in HCS 2000

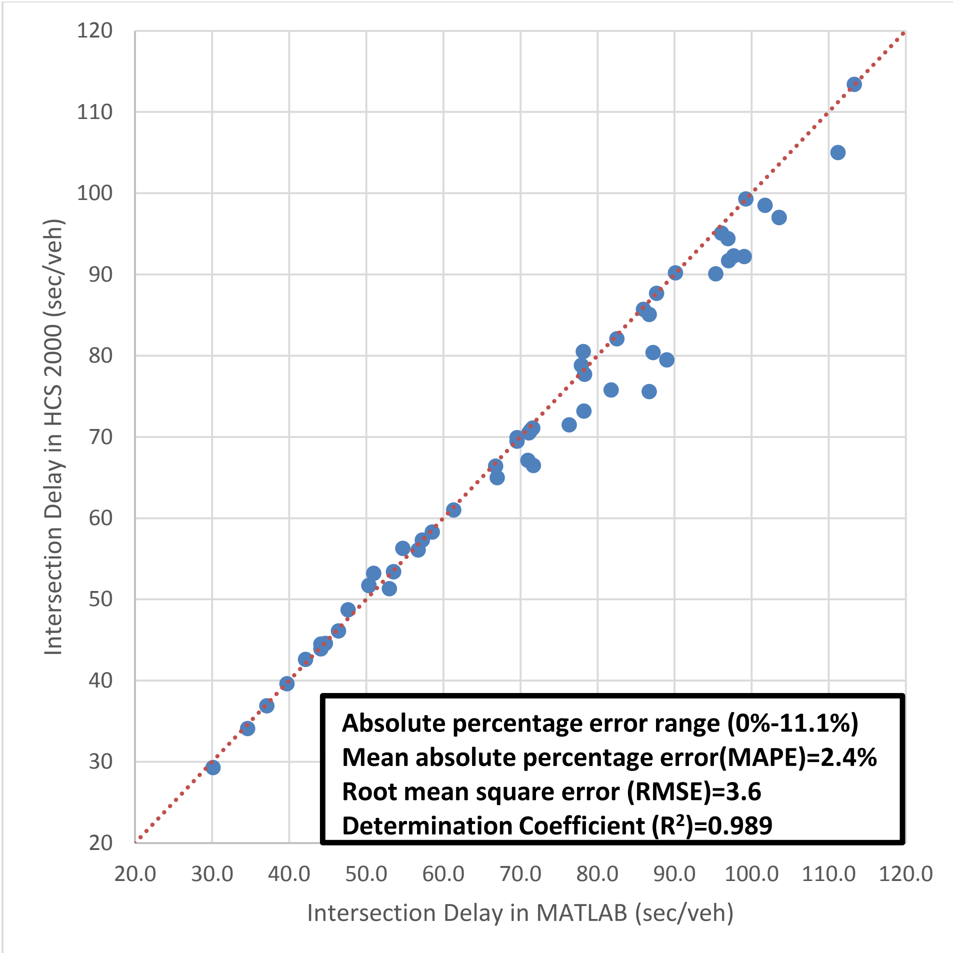


Figure 5.13: Intersection delay comparison between MATLAB and HCS 2000

CHAPTER SIX

6 CONCLUSIONS & RECOMMENDATIONS

The last chapter is divided into two sections. First section addresses the conclusions of this research. The second part draws the recommendations that will enhance better outcomes if this research is being applied. Moreover, this section will identify the future work that can be used to develop the model.

6.1 Conclusions

In this study, an evaluation for the effectiveness of applying DLG strategy with optimized signal timing plans for all approaches of signalized intersection is investigated. A model is developed to identify the optimal LGC for all approaches and the optimum signal timing parameters for the whole intersection. High traffic demand variations (spatial variation) during the day degrade the performance of the signalized intersection. The degradation of the performance happens due to the waste of the time-space allocation of the available resources. The new technique improves significantly the mobility and performance for that intersection since it contains a better allocation of the available time-space resources. The new allocation of the time-space resources leads to reduce the intersection delay D_a. The minimum D_a is used to address the optimal LGC and optimum cycle length at isolated intersection. The principle of equal saturation flow rate was taken in consideration whenever the shared lanes are considered.

Different statistical analysis for the results of a typical 4-lanes approaches and 3-lanes approaches was presented. Compared to FLG, DLG reduced the average intersection delay and the cycle length.

A comparison between DLG and FLG is presented to investigate the benefits of applying DLG at a signalized intersection. The results show that the proposed DLG strategy can potentially achieve significantly better performance in terms of optimized cycle length and average intersection delay. For example, for the two approaches case, the average reduction of intersection delay is 65.8 % and the average reduction of the optimized cycle length is 51.0 % when the spatial variation of demand is changes from 5.0 % to 85.0 % with increment of 5.0 % for both WA and EA approaches. The evaluation of the DLG strategy using well-known traffic software confirms the findings from the numerical analysis. The DLG strategy can provide significant energy/environmental benefits since the D_a is decreased.

6.2 Recommendations

The applicability of dynamic lane grouping in real life practice and expected safety and operational problems are considered the major problems of this new technique. Variable Massage Signs (VMS) can be used to overcome the safety issues dealing with applying the new technique by providing the drivers with real time information about the existing lane group combination at the targeted intersection. But still further studies are required, as an extension of this study, to find the best information type which should be delivered to the drivers and the optimal location for VMS considering the required distance for the driver to make the decision before reaching the intersection approaches. Furthermore, the changing in the LGC should be studied carefully in order to reduce the confusion for the drivers.

Further studies for different kind of phasing are required to develop the model, but as mentioned earlier in the problem statement separate phasing is used in Saudi Arabia. Also the delay equation can be modified to account the initial queuing but it requires additional studies.

Another study about the pre-signal is used as a filtering for the traffic movement with some detectors to count the number of vehicles. This kind of signal will help to keep the signal more responsive for the demand.

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Appendix

MATLAB Main Code

```
% Load the defined volume for each approach
loadLanesVolumData;

% Load the rest defined variables
loadOtherVariablesData();

totalApproachVolum = 1000;
lanesDelay = {1:4};
allDelay = (1:4);
fourLanesGroup = {laneGroup1, laneGroup2, laneGroup3, laneGroup4,
laneGroup5, laneGroup6, laneGroup7, laneGroup8};
threeLanesGroup = {threeLanesGroup1, threeLanesGroup2,
threeLanesGroup3, threeLanesGroup4, threeLanesGroup5,
threeLanesGroup6};
minMaxCR = [];
minMaxCR_Print = [];
minMinID_Print = [];
IDFromMinMaxCR = [];
minMinID = [];
CRFromMinMinID = [];
index = 0;
index2 = 0;
zeorsIndex = 0;
cycleLengthDrawData_ID = [];
laneGroupsDrawData_ID = [];
cycleLengthDrawData_CR = [];
laneGroupsDrawData_CR = [];
CPFromMinMinID = [];

        minCapacityRatio = 1000000;
        %minIntersectionDelay = 1000000;
        minCapacityRatioLane = 1;
        minIntersectionDelayLane = 1;
        index = index + 1;
        index2 = index2 + 1;
        lanesGroupMaxCapacitRation = [];
        lanesGroupMinIntersectionDelay = [];
        lanesGroupCombination1 = {};
        lanesGroupCombination2 = {};
        lanesGroupCombination3 = {};
        lanesGroupCombination4 = {};
        lanesGroupCL = [];
        lanesGroupCR1 = {};
        lanesGroupCR2 = {};
        lanesGroupCR3 = {};
        lanesGroupCR4 = {};
        lanesGroupGREE1 = [];
        lanesGroupGREE2 = [];
        lanesGroupGREE3 = [];
        lanesGroupGREE4 = [];
```

```
compinations = [];
leftVolume1 = [];
throughVolume1 = [];
rightVolume1 = [];
leftVolume2 = [];
throughVolume2 = [];
rightVolume2 = [];
leftVolume3 = [];
throughVolume3 = [];
rightVolume3 = [];
leftVolume4 = [];
throughVolume41 = [];
rightVolume4 = [];
finalIndex = 0;
totalApproach1Volum = 1500;
totalApproach2Volum = 474;
totalApproach3Volum = 1500;
totalApproach4Volum = 87;
%skipThisLaneGroup = false;

for i1 = 0.05:0.05:0.85
    leftVolum1= floor(i1 * totalApproach1Volum);
    for j1 = 0.9-i1
        throughVolum1 = floor(j1 * totalApproach1Volum);
        rightVolum1 = 150;

      for i2 = 160
         leftVolum2= i2;
         for j2 = 36
            throughVolum2 = j2;
           rightVolum2 = 278;

      for i3 = 0.05:0.05:0.85
       leftVolum3 = floor(i3 * totalApproach3Volum);
                    for j3 = 0.9-i3
            throughVolum3 = floor(j3 * totalApproach3Volum);
                        rightVolum3 = 150;

                         for i4 = 42
                            leftVolum4 = i4;
                                for j4 = 20
                                    throughVolum4 = j4;
                                    rightVolum4 = 25;

                                    compIndex = 0;
                                    %TEMPleftVolume4 = [];
                                   %TEMPthroughVolume4 = [];
                                    %TEMPrightVolume4 = [];

                                    TEMPlanesGroupCR1 = {};
                                    TEMPlanesGroupCR2 = {};
                                    TEMPlanesGroupCR3 = {};
                                    TEMPlanesGroupCR4 = {};
                                   TEMPlanesGroupGREE1 = [];
                                   TEMPlanesGroupGREE2 = [];
                                   TEMPlanesGroupGREE3 = [];
```

```
                                        TEMPlanesGroupGREE4 = [];
                                         TEMPcompinations = [];
                                         TEMPlanesGroupCL = [];

TEMPlanesGroupMinIntersectionDelay = [];

                         for app1Lanes = 1:8

                group1 = fourLanesGroup{app1Lanes};
 [group1, shared, skip] = sharedLaneFunction(group1, leftVolum1,
throughVolum1, rightVolum1);
  if shared == false
            group1 = volumeDistributionFunction( group1, leftVolum1,
throughVolum1, rightVolum1 );
                                                end
    if skip == false
                        for app2Lanes = 1:1
          group2 = threeLanesGroup{app2Lanes};
   [group2, shared, skip] = sharedLaneFunction(group2, leftVolum2,
throughVolum2, rightVolum2);

         if shared == false
                                                                 group2 =
volumeDistributionFunction( group2, leftVolum2, throughVolum2,
rightVolum2 );
                                                     end
           if skip == false
           for app3Lanes = 1:8

group3 = fourLanesGroup{app3Lanes};

[group3, shared, skip] = sharedLaneFunction(group3, leftVolum3,
throughVolum3, rightVolum3);

         if shared == false

group3 = volumeDistributionFunction( group3, leftVolum3,
throughVolum3, rightVolum3 );
                                                              end
        if skip == false

for app4Lanes = 1:1

group4 = threeLanesGroup{app4Lanes};

[group4, shared, skip] = sharedLaneFunction(group4, leftVolum4,
throughVolum4, rightVolum4);


if shared == false

group4 = volumeDistributionFunction( group4, leftVolum4,
throughVolum4, rightVolum4 );

end
```

```
if skip == false

compIndex = compIndex + 1;

allVolums = {group1, group2, group3, group4};

[calc, CL, CR1, CR2, CR3, CR4, GREE1, GREE2, GREE3, GREE4] =
calculations(allVolums);

tempCapAndDel = calc;

comText = strcat(num2str(app1Lanes), num2str(app2Lanes),
num2str(app3Lanes), num2str(app4Lanes));

str = sprintf('%d', comText);

TEMPcompinations{compIndex} = comText;

TEMPlanesGroupMaxCapacitRation(compIndex) = tempCapAndDel(1);

TEMPlanesGroupMinIntersectionDelay(compIndex) = tempCapAndDel(2);

TEMPlanesGroupCL(compIndex) = CL;

TEMPlanesGroupCR1{compIndex} = CR1;

TEMPlanesGroupCR2{compIndex} = CR2;

TEMPlanesGroupCR3{compIndex} = CR3;

TEMPlanesGroupCR4{compIndex} = CR4;

TEMPlanesGroupGREE1(compIndex) = GREE1;

TEMPlanesGroupGREE2(compIndex) = GREE2;

TEMPlanesGroupGREE3(compIndex) = GREE3;

TEMPlanesGroupGREE4(compIndex) = GREE4;

end

end
                                                                end
                                                            end
                                                        end
                                                    end
```

```
                                            end
                                        end

                                        finalIndex = finalIndex
+ 1;

[miIDValue,minMinID_Index] =
min(TEMPlanesGroupMinIntersectionDelay);

lanesGroupMinIntersectionDelay(finalIndex) = miIDValue;

                                        compinations{finalIndex}
= TEMPcompinations{minMinID_Index};

                                        lanesGroupCL(finalIndex)
= TEMPlanesGroupCL(minMinID_Index);


lanesGroupCR1{finalIndex} = TEMPlanesGroupCR1{minMinID_Index};

lanesGroupCR2{finalIndex} = TEMPlanesGroupCR2{minMinID_Index};

lanesGroupCR3{finalIndex} = TEMPlanesGroupCR3{minMinID_Index};

lanesGroupCR4{finalIndex} = TEMPlanesGroupCR4{minMinID_Index};


lanesGroupGREE1(finalIndex) = TEMPlanesGroupGREE1(minMinID_Index);

lanesGroupGREE2(finalIndex) = TEMPlanesGroupGREE2(minMinID_Index);

lanesGroupGREE3(finalIndex) = TEMPlanesGroupGREE3(minMinID_Index);

lanesGroupGREE4(finalIndex) = TEMPlanesGroupGREE4(minMinID_Index);

leftVolume1(finalIndex) = leftVolum1;
leftVolume2(finalIndex) = leftVolum2;
leftVolume3(finalIndex) = leftVolum3;
leftVolume4(finalIndex) = leftVolum4;


throughVolume1(finalIndex) = throughVolum1;

throughVolume2(finalIndex) = throughVolum2;

throughVolume3(finalIndex) = throughVolum3;

throughVolume4(finalIndex) = throughVolum4;

rightVolume1(finalIndex) = rightVolum1;
rightVolume2(finalIndex) = rightVolum2;
rightVolume3(finalIndex) = rightVolum3;
rightVolume4(finalIndex) = rightVolum4;

                                    end
```

```
                                end
                            end
                        end
                    end
                end
            end
            end

        %finalIndex
        %lanesGroupMinIntersectionDelay

        %[minCapacityRatioLaneValue,minMaxCR_Index] =
min(lanesGroupMaxCapacitRation);
        %[miIDValue,minMinID_Index] =
min(lanesGroupMinIntersectionDelay);
        %MinimumInterSectionDelay = miIDValue;

         data1 = [lanesGroupMinIntersectionDelay',
str2double(compinations)', leftVolume1', throughVolume1',
rightVolume1', leftVolume2', throughVolume2', rightVolume2',
leftVolume3', throughVolume3', rightVolume3', leftVolume4',
throughVolume4', rightVolume4', lanesGroupCL',
lanesGroupGREE1',lanesGroupGREE2',lanesGroupGREE3',lanesGroupGREE4'
];


        filename1 = 'minimumIntersectionDelayData1.xlsx';
        xlswrite(filename1, data1, 1);


ylabel('Percentage of through vehicles');
zlabel('Maximum Capacity Ratio (V/c)');

colorbar

figure;
x = cycleLengthDrawData_ID(:,1);
y = cycleLengthDrawData_ID(:,2);
z = cycleLengthDrawData_ID(:,3);

a=size(cycleLengthDrawData_ID);
b=a(:,1);
xlin=linspace(min(x),max(x),b);
ylin=linspace(min(y),max(y),b);
[X,Y]=meshgrid(xlin,ylin);
Z = griddata(x,y,z,X,Y);
surf(X,Y,Z);
zlim([50 100]);
xlabel('Percentage of left turn vehicles');
ylabel('Percentage of through vehicles');
zlabel('Cycle Length (s)');
colorbar
```

```
figure;
x = laneGroupsDrawData_ID(:,1);
y = laneGroupsDrawData_ID(:,2);
z = laneGroupsDrawData_ID(:,3);

a=size(laneGroupsDrawData_ID);
b=a(:,1);
xlin=linspace(min(x),max(x),b);
ylin=linspace(min(y),max(y),b);
[X,Y]=meshgrid(xlin,ylin);
Z = griddata(x,y,z,X,Y);
surf(X,Y,Z);

xlabel('Percentage of left turn vehicles');
ylabel('Percentage of through vehicles');
zlabel('Lane Group Number');
colorbar;



%%%%

figure
x = minMaxCR(:,1);
y = minMaxCR(:,2);
z = minMaxCR(:,3);

a=size(minMaxCR);
b=a(:,1);
xlin=linspace(min(x),max(x),b);
ylin=linspace(min(y),max(y),b);
[X,Y]=meshgrid(xlin,ylin);
Z = griddata(x,y,z,X,Y);
surf(X,Y,Z);
zlim([.5 1]);
xlabel('Percentage of left turn vehicles');
ylabel('Percentage of through vehicles');
zlabel('Maximum Capacity Ratio (V/c)');
colorbar;



figure
x = IDFromMinMaxCR(:,1);
y = IDFromMinMaxCR(:,2);
z = IDFromMinMaxCR(:,3);

a=size(IDFromMinMaxCR);
b=a(:,1);
xlin=linspace(min(x),max(x),b);
ylin=linspace(min(y),max(y),b);
[X,Y]=meshgrid(xlin,ylin);
Z = griddata(x,y,z,X,Y);
surf(X,Y,Z);
zlim([20 60]);
```

```
xlabel('Percentage of left turn vehicles');
ylabel('Percentage of through vehicles');
zlabel('Average Intersection Delay (s/v)');
colorbar


figure;
x = cycleLengthDrawData_CR(:,1);
y = cycleLengthDrawData_CR(:,2);
z = cycleLengthDrawData_CR(:,3);

a=size(cycleLengthDrawData_CR);
b=a(:,1);
xlin=linspace(min(x),max(x),b);
ylin=linspace(min(y),max(y),b);
[X,Y]=meshgrid(xlin,ylin);
Z = griddata(x,y,z,X,Y);
surf(X,Y,Z);
zlim([50 100]);
xlabel('Percentage of left turn vehicles');
ylabel('Percentage of through vehicles');
zlabel('Cycle Length (s)');
colorbar


figure;
x = laneGroupsDrawData_CR(:,1);
y = laneGroupsDrawData_CR(:,2);
z = laneGroupsDrawData_CR(:,3);

a=size(laneGroupsDrawData_CR);
b=a(:,1);
xlin=linspace(min(x),max(x),b);
ylin=linspace(min(y),max(y),b);
[X,Y]=meshgrid(xlin,ylin);
Z = griddata(x,y,z,X,Y);
surf(X,Y,Z);

xlabel('Percentage of left turn vehicles');
ylabel('Percentage of through vehicles');
zlabel('Lane Group Number');
colorbar;
```

Vitae

Name	: Muath Marwan Najjar
Nationality	: Palestinian
Date of Birth	: 16/8/1988
Email	: maaznajjar@gmail.com
Address	: Nablus. Palestine.
Academic Background	: Bachelor in Civil engineering, An-Najah National University. Nablus Palestine. With a graduation project named by Simulation-based analysis of traffic condition in Nablus CBD and options for improvement